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PROCEEDINGS

OF THE

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VOL. 74 NOVEMBER, 1948 No. 9

TECHNICAL PAPERS

AND

DISCUSSIONS

A list of "Current Papers and Discussions" may be found on the page preceding the table of contents

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	Published	Discussion closes
Bergendorff, R. N., and Sorkin, Josef. Mississippi River Bridge at Dubuque, Iowa	June, 1947	
Discussion in Sept., Oct., 1947, May, 1948		Closed*
Moore, William W. Experiences with Predetermining Pile Lengths	Nov., 1947	
Discussion in Mar., June, 1948		Closed*
Turner, Robert E. Operation of the Conowingo Hydroelectric Plant	Nov., 1947	
Discussion in June, Sept., 1948		Closed*
Hickerson, T. F. Determination of Position and Azimuth by Simple and Accurate Methods	Nov., 1947	
Discussion in June, Nov., 1948		Closed*
Bell, S. J. A Centroidal Method of Rigid-Frame Analysis	Nov., 1947	
Discussion in June, Sept., 1948		Closed*
Loring, Samuel J. Experimental Determination of Vibration Characteristics of Structures	Dec., 1947	
Discussion in Mar., June, Sept., 1948		Closed*
Chen Pei-ping. Matrix Analysis of Pin-Connected Structures	Dec., 1947	
Discussion in June, Oct., 1948		Closed*
Andrew, Charles E. Unusual Design Problems—Second Tacoma Narrows Bridge	Dec., 1947	
Discussion in June, 1948		Closed*
Stewart, Ralph W. Analysis of Frames with Elastic Joints	Dec., 1947	
Discussion in May, June, Sept., 1948		Closed*
Levinton, Zusse. Elastic Foundations Analyzed by the Method of Redundant Reactions	Dec., 1947	
Discussion in Sept., Oct., 1948		Closed*
Symposium: Lateral Earth Pressures on Flexible Retaining Walls	Jan., 1948	
Discussion in June, Sept., Oct., 1948		Closed*
Neiman, A. S. Shearing Stress Distribution in Box Girders with Multiple Webs	Feb., 1948	
Discussion in June, Sept., 1948		Closed*
Harza, L. F. The Significance of Pore Pressure in Hydraulic Structures	Dec., 1947	
Discussion in June, Sept., Oct., Nov., 1948		Dec. 1, 1948
Symposium: Highway Bridge Floors	Mar., 1948	
Discussion in Sept., Oct., Nov., 1948		Dec. 1, 1948
DuVal, Miles P. The Marine Operating Problems, Panama Canal, and the Solution	Feb., 1947	Dec. 1, 1948
Claybourn, J. G. Sea Level Plan for Panama Canal	Feb., 1947	
Discussion in Feb., June, 1948		Dec. 1, 1948
Symposium: Panama Canal—The Sea-Level Project	Apr., 1948	
Discussion in June, Sept., Oct., Nov., 1948		Dec. 1, 1948
Chellis, Robert D. The Relationship Between Pile Formulas and Load Tests	May, 1948	
Discussion in Sept., Nov., 1948		Dec. 1, 1948
Wimer, H. Dudley, Jr. Stability of Thin-Walled Compression Members	May, 1948	
Discussion in Nov., 1948		Dec. 1, 1948
Scharff, Maurice R. Valuation and Depreciation of Public Utility Property	June, 1948	
Discussion in Oct., Nov., 1948		Jan. 1, 1949
Report. Review of Slope Protection Methods: Report of the Subcommittee on Slope Protection of the Committee on Earth Dams of the Soil Mechanics and Foundations Division	June, 1948	
Discussion in Oct., 1948		Jan. 1, 1949
Symposium: Industrial Wastes	Sept., 1948	Feb. 1, 1949
Chu, Chung-jui. Adjustment of the "Shoran" Triangulation	Sept., 1948	Feb. 1, 1949
Feil, Louis G. Unusual Design Problems, Harlan County Dam	Sept., 1948	Feb. 1, 1949
Anderson, A. A. Expansion Joint Practice in Highway Construction	Sept., 1948	Feb. 1, 1949
Tsien, Ling-hi. A Simplified Method of Analyzing Suspension Bridges	Sept., 1948	Feb. 1, 1949
Tsai, Fang-Yin. Flexural Constants of Haunched Beams by Area Computation	Sept., 1948	Feb. 1, 1949
Bleich, Friedrich. Dynamic Instability of Truss-Stiffened Suspension Bridges Under Wind Action	Oct., 1948	Mar. 1, 1949
Report. Advances in Sewage Treatment and Present Status of the Art: Fourth Progress Report of the Committee of the Sanitary Engineering Division on Sewerage and Sewage Treatment	Oct., 1948	

NOTE.—The closing dates herein published are final except when names of prospective discussers are registered for special extension of time.

* Publication of closing discussion pending.

CONTENTS FOR NOVEMBER, 1948

PAPERS

PAGE

The Logical System of Freight Rates.
By Charles L. Hall 1427

The Municipal Asphalt Plant, Borough of Manhattan.
By L. Csanyi 1433

Duluth-Superior Harbor.
By Heston R. Cole 1459

Some Effects of Anaerobic Digestion on Sewage Sludge.
By A M Rawn and E. J. Candel 1467

Mineral Land Surveys.
By Arthur J. McNair 1481

DISCUSSIONS

Panama Canal—The Sea-Level Project: A Symposium.
By George B. Massey, William Allan, and Boris A. Bakhmeteff 1509

Highway Bridge Floors: A Symposium.
By Reto Furrer 1518

Stability of Thin-Walled Compression Members.
By Boris Bresler, and George Winter 1523

The Significance of Pore Pressure in Hydraulic Structures.
By Frederick L. Hotes, R. W. Carlson and Raymond E. Davis, and W. G. Morrison 1531

The Relationship Between Pile Formulas and Load Tests.
By Jacob Feld, Richard F. Kafka, G. B. R. Pimm, and George D. Hartley 1539

CONTENTS FOR NOVEMBER, 1948 (*Continued*)

PAGE

Valuation and Depreciation of Public Utility Property.
By W. V. Burnell 1545

Determination of Position and Azimuth by Simple and Accurate Methods.
By T. F. Hickerson 1549

GUIDEPOST FOR TECHNICAL READERS NOVEMBER, 1948, *PROCEEDINGS*

Items of value or significance to readers in various fields are here listed. For convenience the distribution is given in terms of the Society's Technical Divisions

Technical Division	*Pages*
City Planning	1427, 1433, 1459, 1481, 1509, 1545
Construction	1433, 1459, 1510, 1531, 1539
Engineering Economics	1427, 1453, 1459, 1545
Highway	1427, 1518
Hydraulics	1509, 1531
Irrigation	1531
Power	1455, 1531
Sanitary Engineering	1433, 1467
Soil Mechanics and Foundations	1481, 1531, 1539
Structural	1518, 1523, 1531, 1539
Surveying	1481, 1549
Waterways	1430, 1459, 1509

(A constant effort is made to supply technical material to Society members, over the entire range of possible interest. In so far as your specialty may be covered inadequately in the foregoing list, this fact is a gage of the need for your help toward improvement.—Ed.)

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

THE LOGICAL SYSTEM OF FREIGHT RATES

BY CHARLES L. HALL,[1] M. ASCE

SYNOPSIS

The effects of the existing illogical system of freight rates are presented, and a plea is made for a new system. The plan calls for a separation, to be reflected in all rates, between terminal charges and actual transportation charges. It further requires (1) a uniform classification of all freight, regardless of its method of carriage; (2) charges for each form of carrier, uniform throughout the United States, fixed by each branch of the transportation industry in terms of the cost of moving the most easily transported product; (3) a ratio between charges by the various types of carrier, to be determined by independent members of the engineering profession; and (4) determination by a public body of the basic rate, to which the foregoing ratios shall apply.

Struck by the difficulty of determining the savings resulting from the introduction of a new form of carrier between any two terminals, the writer has analyzed the existing system of fixing freight rates. This analysis has convinced him that the public can be fully served only if the whole freight rate structure is established on a purely rational basis without regard for what might be called the "particularistic" motives on which the existing system of rates is based.

The United States has been built up by internal free trade. Thus, any commodity can be moved from any point within the country to any other point within the country without consideration of either customs duties or the relative values of money, as there are no customs and the value of money is uniform. Moreover, the cost of the sale or the distribution of an article, once it arrives at its destination, is independent of its place of manufacture. No one claims that the United States could have been built up on any other system. Admittedly, however, the price of the article, when sold to the ultimate consumer, must include the cost of transporting it from its point of production to its point of consumption.

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **April 1, 1949.**

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From the standpoint of national prosperity and union, the point to be selected as a manufacturing or processing site should be the place where the necessary raw materials can be most economically assembled and processed, and where the total cost of transporting the finished products to points of consumption is a minimum. This is, more or less, how a manufacturer selects a site. If the cost of assembling raw materials and transporting the products thereof is based entirely on the necessary expenses of transportation, all sections of the United States would be treated fairly. Each area of the country would be engaged in the business it can most economically pursue.

If, as is now the case, the cost of transporting any particular raw materials or manufactured products is due to any plan for putting more than a fair share of transportation cost on a few items, or on items manufactured in one section of the United States, in order to permit less than a fair share of such a cost to be allotted to some other items, or to items from some other part of the country, somebody is being treated unfairly. The economic result to the processor or manufacturer is the same as that he would suffer if duties were charged on imports, or if his dollar were depreciated. The system under which the country has been built up would then have been altered, and the national union weakened.

These truisms have scarcely ever been denied. Discussion of details nearly always has been based on the definition of "fair." Few members of the engineering profession have been heard from. This paper is an attempt to apply engineering principles to the question of freight rates.

Of course commodities to be moved are not identical. Nobody expects transits to be moved by any carrier as cheaply as crushed rock. From the standpoint of pure logic, freight should be classified according to the difficulties involved in loading, transporting, delivering, and protecting it. This latter factor will allow some higher classifications for more expensive products. The obvious first step in regulation of freight rates is the classification of all forms of material into as few groups (or classes) as possible. The classification must be applicable to all forms of carriers, with the exception of some products which are physically unsuited to some carriers—for example, very few types of freight can be carried in pipe lines. Subject to that limitation, however, there should be a general classification of freight, agreed to by representatives of all the carriers in the United States. Normally they should agree. If they do not, the Interstate Commerce Commission will have to do the work.

When a package is moved from point A to point B, four different services are performed. Somebody loads the package into a vehicle at A, moves the vehicle (with the package in it) from A to B, unloads the package at B, and meanwhile protects it from weather and thieves. The costs incurred in loading the package are, in the normal case, independent of its destination and are due solely to the physical characteristics of the package, its volume, its mass, and the number of packages of the same sort, intended for the same vehicle, committed to it at the same time. The cost of transporting the vehicle is due to its weight, the distance it travels, the amount of uphill haul involved, and—in unbalanced traffic—the distance the vehicle must subsequently travel unloaded to secure another revenue producing load. The cost of unloading it depends

on the same factors as does the cost of loading. The cost of protection varies with packaging, perishability, and value.

These costs cannot, of course, be isolated for any particular package, because there are too many variables. Certain terminals are more efficient than others. It is usually cheaper to move a vehicle a given number of miles on a much-traveled road than on a poorly-traveled one, because there is a vast amount of road maintenance and central operation partly independent of train density. Taxes vary from state to state. Nevertheless, although the cost of moving any particular package cannot be determined, it is possible to determine—on a national basis—the relative costs of loading (or unloading) a package of one type and a package of another. Similarly, it is possible to compare the cost of transporting a vehicle of one type and gross weight for a given number of miles with the cost of moving a vehicle of another type and gross weight for the same number of miles. It is even possible to compare the relative costs of moving vehicles where there is much uphill haul, and where there is none. The costs of protection can be divided into protection connected with loading and unloading, and protection in transit.

Let an algebraic symbol, a, be assumed as the lowest unit cost. This can perhaps be taken as the cost of transporting 1 ton of coal for 1 mile on a railway of no grade, the coal being moved as part of a carload lot consigned at one point by one consignor to another point for one consignee. It should then be possible to state, as a multiple of a, every other factor connected with every other cost. For example, take a ton of household goods, moving as part of a carload lot. The loading cost might be 450 a per ton; the ton-mile cost, 4 a; the elevation cost, based on the number of feet of increases in elevation on all the uphill grades, 2 a per ton-foot; and the unloading cost, 390 a per ton.

In actual practice, of course, very few cars, except those moving less than carload lot shipments, are loaded or unloaded by the carrier. It is desirable, however, to retain the loading and unloading charge to cover the few cases, like transfers from rail to truck carriers, where it is incurred. Moreover, some part (perhaps 15%) of the loading fee should be charged against the shipper under any circumstances, because the carrier accumulates at terminals some charges which cannot equitably be assessed on any sort of mileage basis. To prevent abuse of terminal facilities there might be a minimum charge per car, regardless of the weight of its contents, to avoid very short carload shipments.

It should be noted that the loading and unloading charges are based on the weight of the packages moved, whereas the ton-mile cost should equitably be based on the weight of the loaded vehicle. This would automatically dispose of arguments about minimum carloadings. If the shipper wanted to pay a rate based on a 50-ton car for moving 1 lb, that would be his privilege. He would, perhaps, have had to pay a minimum terminal fee, as indicated in the preceding paragraph. Naturally, the surcharge on uphill traveling also should be based on tare weight. Carload lot tariffs would thus require only three subheads under each classification of freight: Loading charge per ton (net), transportation per ton-mile (gross), uphill haul per ton-foot (gross). It would probably be necessary to prescribe that the distance (and the uphill grade) of the most advantageously situated carrier should govern all movements be-

tween any two given cities. Otherwise, a carrier with a few miles shorter haul would monopolize all the traffic. However, a carrier should be allowed to demand rates based on its actual mileage if it is willing to risk losing traffic.

The fourth factor governing rates (value of the freight) can be handled in several ways. Probably the most convenient is to assume certain maximum values per ton under each classification, to be eventually reflected in the ton-mile rate for that class of freight, and to require any excess risks to be covered by insurance. This is more or less the method of compensating for the liability now.

Shipments of less than carload lots would naturally involve more factors. There presumably would be entries in the freight rate book under each classification for loading rate per hundredweight, loading rate per individual package, corresponding unloading rates, cost per hundredweight-mile, and cost per hundredweight-foot for uphill traffic. Needless to mention, weights used would be net. Presumably there would be a prescribed minimum weight for each shipment, so that the carriers would not be annoyed by small packages; or, if they were so annoyed, they would be paid for their trouble.

The preparation of the rate book, in terms of a and not of dollars, should be committed to representatives of the railroads. Presumably these would vote according to the ton-milage of their respective roads. Once all local controversies about special rates are eliminated, as they will be in this plan, the railroad industry can be relied on to settle these matters better than anyone else. Its chiefs are well aware of the relative costs of moving different kinds of goods. Nobody in the industry should have a motive for unbalancing such data for sordid reasons. If he had, his colleagues could soon find him out.

A similar plan must be followed for inland river commerce. This branch of transportation is in the ridiculous situation of charging at the same rate for upstream and downstream commerce, although everyone knows that the most economic balance is 8 lb downstream to 3 lb upstream. Whereas the economy of bargeload lots should be recognized, it probably would not be necessary to quote any differences between small lot shipments and bulk shipments, so that the significant entries in the water rate book would number eight—classification of freight, loading charge per package, loading charge per ton, corresponding unloading charges, transportation per ton-mile, percentage of reduction in this value for bargeload lots, allowable weight of maximum package, and dimensions of maximum package. The table of distances would merely show water distances between all points. To penalize upstream traffic, distances upstream should count double. Thus, the present unremunerative balance would quickly be destroyed.

The data cited could not of course be absolute. They would have to be quoted in terms of some other algebraic symbol b, which might be defined as the cost of carrying 1 ton of coal 1 mile downstream.

The rates should be based on the expectation that, at rail-to-water transfer points, the shipper would have to pay the rail unloading charge or the water loading charge, whichever were the greater. By application of this rule, joint rail-water rates would not be difficult to determine.

The maritime industry should decide the costs of movement in terms of b just as previously suggested for the railroads. There would appear to be no motive either to prescribe an unrealistic schedule or to play false with the public.

Exactly similar arrangements should be made for coastwise and Great Lakes shipping, common carrier trucks, pipe lines, and—if deemed wise—air traffic. The transportation industry would thus finally have a series of values, a, b, c, . . . , n, independent of each other. Obviously, the next problem is to fix a ratio such that $b = p\,a$, $c = q\,a$, and so forth. The evaluation of p, q, r, . . . is a problem in engineering and should be committed to a group of civil and mechanical engineers, either permanent members of the Civil Service or engineers having no financial interest in any branch of the transportation industry. The object in setting these ratios is to cause traffic to move along the most economical lines, so that the real cost to the citizens of the United States for moving a unit of freight through a unit of distance will be a minimum. The ratios certainly cannot be formulated to scare branches of transportation into reducing individual rates, for which purpose too much competing traffic is now being used.

There still remains the basic problem of determining the value of a—essentially a mixed problem of finance and politics, which would have to be accomplished by a first-class body responsible in the last analysis to the people (presumably the Interstate Commerce Commission). The task of this body would be to fix the minimum value under which the American public would be served and which, at the same time, would provide a reasonable return for the capital employed in transportation, taken as a whole. There would be little temptation either to fix the amount too high, for that would bring commerce to a standstill, or to fix it too low, for that would bankrupt the carriers. The quality required is good judgment rather than the ability to resist the pressure of special interests.

In this connection it should be noted out that a subsidy to any carrier, or its operation with public assistance at an admitted loss, unbalances the whole setup. For instance the granting of any form of subsidy to an ocean carrier moving from Sacramento, Calif., to Houston, Tex., through the Panama Canal would result in the movement of certain goods by water between the Pacific Coast and Texas which ought to have moved by rail. Thus, the value of the ratio between rail and ocean carrier rates is reduced artificially. If the carriers are operating at a loss in the first place, the increased traffic by water, caused by the relative rate reductions, either overloads the subsidy and thus harms the taxpayer, or deprives the railroad of freight and thus drives the rail-water ratio further out of balance. Most likely it does both. Only if carriers meet all their own expenses out of their own receipts will the logical system of freight rates work smoothly.

It can thus be noted that there are four successive steps in establishing a logical and noncontroversial system of freight rates:

(1) The classification of freight according to the costs of handling it;

(2) The establishment of appropriate relative costs within each type of transportation, to be accomplished by a body representing the industry;
(3) The establishment of ratios between the basic costs of the various forms of transportation, to be accomplished by a body representing the engineering profession; and
(4) The establishment of a basic unit cost of national transportation, to be accomplished by a body representing the public.

The first task (classification of freight) is easy if the carriers by rail, water, truck, and pipe line can get along with each other. Even if they do not, the authority charged with adjusting differences should not have too many difficulties. It should always be possible to transfer goods from one class to another by unanimous agreement of all carriers. Increase in the number of classifications or contested changes in classification should be made very difficult.

The second task (rate schedules for the various types of carrier) is perhaps the most difficult, but the results should be relatively permanent when once accomplished, as seldom will any invention make it possible to carry some particular form of goods at an unexpectedly low rate. Revision at 10-year intervals should be sufficient.

The third task (fixing the ratios) must be performed from time to time. Inventions effecting economies in one form of transportation, but not in all forms, have been frequent during the life of the United States. Revision at 5-year intervals probably will be required.

The fourth task (fixing the basic transportation cost) will be almost continuous, for the general level of transportation costs is altered almost as fast as the cost of living. The Interstate Commerce Commission will probably have to hold hearings on the subject every year.

The advantages of this scheme, taken as a whole, are obvious. The interminable squabbles between sections of the United States, and between ports of entry, are abolished. The country will be better integrated. Transportation in general is assured a reasonable return. All freight will be moved by the carrier best suited to the task and the nation will be benefited thereby.

The objections to the plan rest basically on its effects on capital already expended. Undoubtedly some lines in every type of transportation will be forced out of business. Certain ports which now have an unfair advantage, based on reasons having nothing to do with engineering facts, will lose commerce. However, the United States has been built up by the ruthless scrapping of outmoded forms of industry. The first railroads to the west ruined both stage coach lines and New England agriculture. Where would the nation be if railroad building had been delayed for any such reason? This plan will put transportation on its mettle. The best argument in its favor is that if the proposed plan were now in force nobody would think of setting up the present illogical rate structure.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

THE MUNICIPAL ASPHALT PLANT, BOROUGH OF MANHATTAN

BY L. CSANYI,[1] M. ASCE

SYNOPSIS

The Municipal Asphalt Plant of the Borough of Manhattan (New York, N. Y.) is situated in a modern, fashionable, residential area. Why such a site was selected is discussed briefly. The various factors involved in the final acceptance of the plant as a good neighbor—such as the character and operation of the equipment and machinery installed, the new operational procedures developed, the adaptation of unique machinery, and the use of large 4-ton batch mixers—are separately developed. Manufacturing costs covering 1 year of operation during 1944 and 1945 are compiled and presented.

The conclusions indicate that the foresight and courage of the planners of the plant, in selecting the site and utilizing the modern equipment, are amply repaid by the efficient and economical operation of the plant.

INTRODUCTION

On the Franklin D. Roosevelt Drive (East River Drive) at East 90th Street in New York, facing the East River and Hell Gate Bridge, stands a building which compels attention by its unusual design (Fig. 1). Overlooking it is Gracie Mansion, residence of the mayor of the City of New York, beside it on two sides are towering apartments with gardens and penthouses, and on the other side is a park playground. This unique building, in this beautiful setting, is the Municipal Asphalt Plant of the Borough of Manhattan, dubbed the "Cathedral of Asphalt" by Robert Moses, Commissioner of the Department of Parks.

On realizing its purpose many questions arise. Why was an asphalt plant built in such a highly developed area? How does it overcome the objectionable characteristics that make other asphalt plants unacceptable in similar areas? Why was this plant built in an area of such high development? The plant is a manufacturing enterprise, and must consequently meet the basic principles

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by April 1, 1949.

[1] With Edwards and Kelcey, Cons. Engrs., New York, N. Y.

of every manufacturing enterprise if it is to be successful. These basic principles require that a ready access to an abundant supply of raw materials be available, that the manufacturing process be as simple and as economical as possible, and that an easy method of distribution of its products to a broad market be provided. Having provided for these fundamentals, it is also neces-

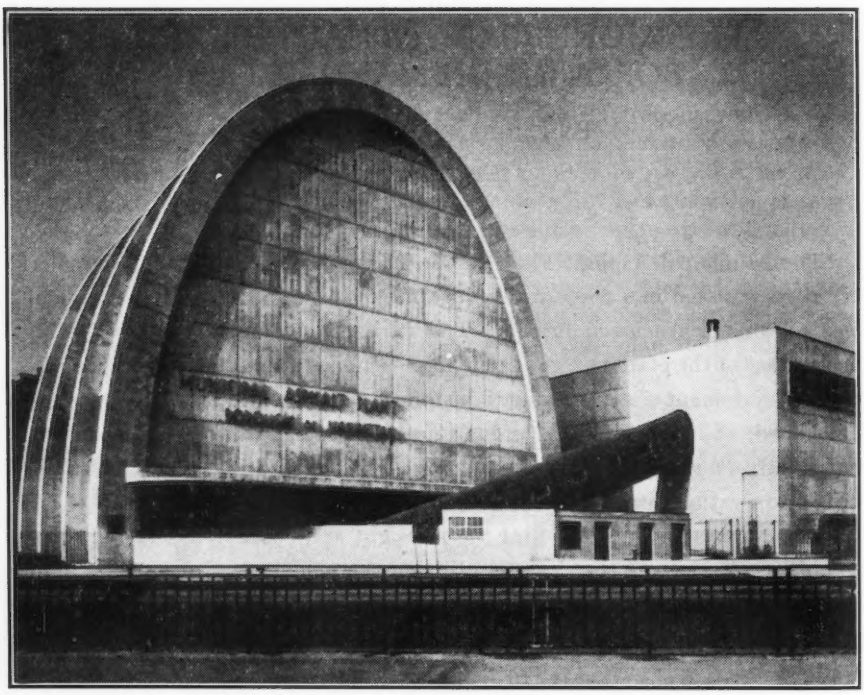

FIG. 1.—NEW MUNICIPAL ASPHALT PLANT, BOROUGH OF MANHATTAN, NEW YORK, N. Y.

sary for them to be maintained in economic equilibrium as ever changing conditions require. The choice of any plant site, therefore, is predicated almost entirely on a balance between the source of raw materials and the distribution of the finished products—with all the ramifications of transportation, handling, and storing of materials involved.

THE PLANT SITE

In the Borough of Manhattan there prevail certain conditions which must be weighed if the balance is to be achieved. There, all the raw materials are predominantly available by barge delivery, making a waterfront site essential. Furthermore, because asphaltic paving mixtures of the hot plant-mix type cannot be stored, it is essential that rapid, comparatively short hauls to the site of their application be provided. In addition, the output of this plant is generally utilized for maintenance and repair work, so that the need for complete, economical coverage of the entire area serviced must also be considered.

Bearing these conditions in mind while glancing at a map of Manhattan, which shows an island some 15 miles in length and varying in width from about ½ mile to 2½ miles, it becomes quite evident that a site on either the East River or the North River between 72d Street and 110th Street would be most satisfactory. Closer examination of these areas discloses that the North River side is unavailable, and that the East River side may be utilized under certain conditions. The North River side cannot be used due to the absence of commerical waterfront areas offshore the Henry Hudson Drive. On the East River side the extension of the East River Drive is proposed through this area to complete another link in the circumferential highway system of the island. However, between 92d Street and 98th Street a commercial area is provided wherein the plant may be located. Furthermore, rezoning of areas along the

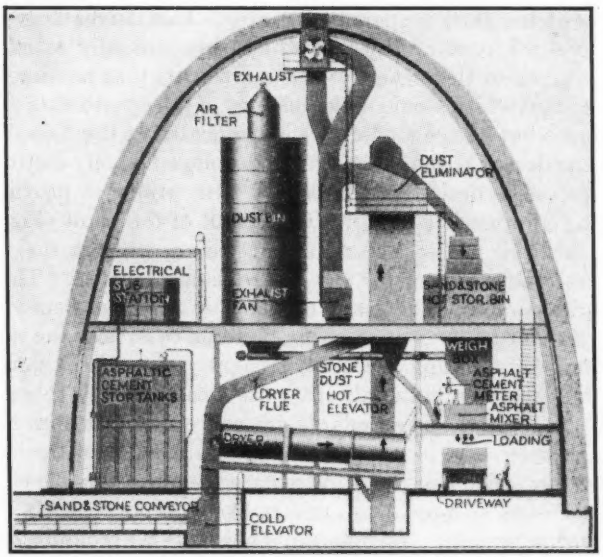

FIG. 2.—INTERIOR OF PLANT

drive is contemplated to reclaim the area into highly restricted districts. The dividing line runs through 90th Street, with provision for adjustment in the unrestricted zone to the north. The old asphalt plant (since razed) was also situated at 90th Street. Thus, this site was retained for the new plant as first choice.

Even though this area would be favorable to most efficient operation, other areas were studied where adjustment of the plant to a highly residential district would not be necessary, and where perhaps some efficiency in distribution could be forfeited to less expensive accommodations than those required at the 90th Street site. In line with this program, sites at 132d Street on the Harlem River and at 32d Street on the East River were studied. These sites were within the proposed commercial areas provided in conjunction with the layouts of the East River Drive and the Harlem River Drive. After exhaustive studies these

sites were rejected for several reasons, among which were the curtailment and the combining of commercial areas along the river front to enhance the beauty of the drive and to yield better efficiency to waterfront operations. To infringe upon already limited waterfront operations of private industry would offset the desired efficiency and further limit barge delivery of materials, not only to private plants, but also to the asphalt plant. Thus, the 90th Street site on the East River was finally selected as the site of the new asphalt plant because of the ideal relationship between the source and the distribution of its materials.

Realizing the anticipated change in the vicinity of the selected site, particularly with regard to the construction of garden apartments on the south side of 90th Street, and to the East River Drive at its door, it became apparent that the conventional asphalt plant could not be used. Conferences between the architects of the garden apartments, of the East River Drive, and of the plant developed a layout and design which was mutually satisfactory. Assurances were given to the owners of the apartments that no annoyance would arise from the plant which would make tenancy of the apartments objectionable. Influenced somewhat by the modern trends indicated at the New York World's Fair (1939), the design of the building was developed wholly distinct from conventional lines. The design blends beauty with utility in providing efficient housing for the equipment (Fig. 2). The layout of the plant (Fig. 3) provides for dockside facilities within a park-like enclosure, wherein the gantry crane and the low receiving hopper for the materials are located. The connection between the dockside and the plant is made by a tunnel beneath the drive. There is also provided a large storage-bin building to enclose the raw materials, thereby not only eliminating the usual unsightly mounds of sand and stone, but also protecting these materials from the weather. By these innovations the plant, as far as aesthetic standards are concerned, becomes acceptable to the residents near by, and blends into the general beauty of the landscape.

The manner in which this plant overcomes the objectionable characteristics of other plants—such as noise, dust, and smoke—involves nearly every step of the manufacturing process. To determine how this is accomplished, each step of the process is reviewed separately.

Raw Materials

Sand and Stone.—The raw materials used by the plant are sand, stone, mineral filler, asphaltic cement, and fuel oil. The sand is obtained under rigid specifications[2] for character and gradation from sources around Port Washington (Long Island) or South River, N. J. The crushed stone, usually a limestone, is similarly secured under specifications[3] from the upper Hudson River Valley. Both materials are delivered alongside the dock area in barges of from 600-cu-yd capacity to 1,000-cu-yd capacity.

The storage-bin building, wherein the materials are stored, contains 5 bins—1 bin of 3,000-cu-yd capacity for asphalt sand; 1 of 2,000 cu-yd capacity for

[2] "Specification for Sand," *24-S-35-S*, Board of Standardization, Dept. of Purchase, City of New York, New York, N. Y., 1945.

[3] "Specification for Aggregate Coarse," *24-A-20-S*, Board of Standardization, Dept. of Purchase, City of New York, New York, N. Y., 1945.

broken stone; 2 of 1,000-cu-yd capacity for concrete, sand, and stone; and 1 coal bin of 200-ton capacity.

The various materials are transferred by a gantry crane from the barge to a hopper, which feeds the main conveyor belt. This belt is located in a tunnel under the East River Drive and carries the material up an enclosed incline to the top of the bins, where it discharges upon a distribution system consisting of reversible cross and transfer conveyors, and three movable, reversible shuttle conveyors that discharge the materials into their respective bins. The entire system has a capacity of 100 tons per hr and is operated by five men—two barge trimmers, one craneman, and two belt operators.

With the exception of the crane, the entire system is enclosed. The only part that is visible is the transfer from the barge to the hopper, where the

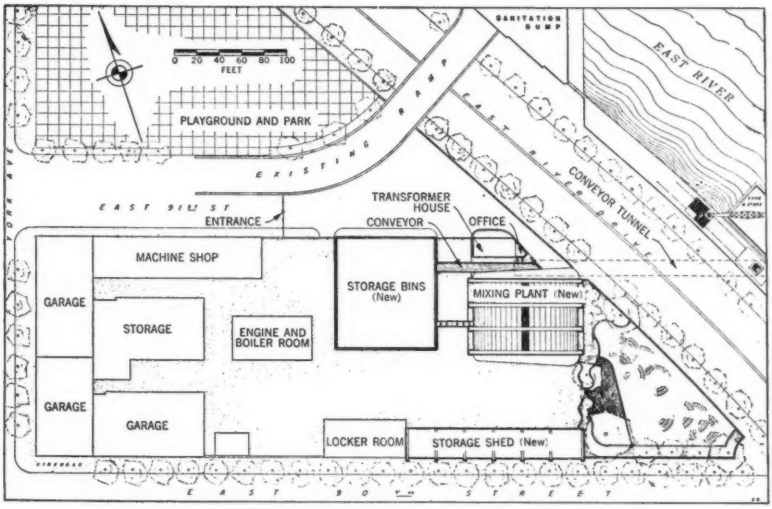

FIG. 3.—LAYOUT PLAN OF PLANT

material seemingly disappears. Inasmuch as the entire system is electrically driven it is virtually noiseless.

Asphaltic Cement.—The asphaltic cement (AC) is received alongside the dock in barges with capacities of from 50,000 gal to 60,000 gal. The dockside is of sufficient length, and the hopper and AC intake are so situated that the unloading of barges of both aggregate and AC may be carried on simultaneously. Steam pumps on the barge, to which steam is furnished by a tug, pump the AC through electrically heated lines to the 5 storage and service tanks (total capacity 120,000 gal), which are situated in the main plant building (Fig. 2).

From the intake the pipe line runs in a pipe gallery underground along the dock to the conveyor tunnel, thence along the wall of the tunnel to the plant building, wherein valving is provided to convey the AC to any one, or several, of the tanks.

The AC pipe lines are constructed of 4-in., continuous, welded steel pipe. Instead of the usual steam jacketing, the pipes are fitted with "calrod" heating

units strapped to the underside of the pipe. The pipe and the heating units are covered by a sheet metal covering and a 2-in. asbestos insulation. Fittings, valves, and other pipe appurtenances are suitably fitted with "calrod" heaters and insulation. Electrical thermostats (manually adjusted) are installed in the line at strategic points for automatic temperature control. Details of these heating elements will be found in the original manuscript on which this paper is based, which is on file in the Engineering Societies Library.[3a]

With the lines heated, the connections between barge and line completed, and the tank distribution valves set, pumping begins. The AC is usually delivered at a temperature[4] ranging from 280° F to 330° F. Within this temperature range the pumping rate varies from 14,000 gal per hr to 15,000 gal per hr for a 60–70 penetration AC. When the temperature falls below 270° F, the pumping rate falls rapidly, a heavy overload is placed on the pump, and a heavy vibration synchronous with the pump pulsations is induced in the pipe line. These vibrations when heavy are dangerous, because welds at fittings and bends may be weakened or broken, although AC has been pumped at a temperature as low as 250° F at a pumping rate of from about 4,000 gal per hr to 5,000 gal per hr. Pumping AC at from 300° F to 310° F seems best, for at this temperature it flows smoothly and rapidly into the tanks with very little vaporization.

Mineral Dust Filler.—The mineral dust filler used at the plant is a powdered limestone or dolomite filler meeting the requirements of the New York City specifications.[5] Transfer of the filler from the barge to the silos in the plant is accomplished by several methods—all of which utilize the pumping of the dust but differ in the manner of delivering the cargo alongside. Mineral dust filler may be received alongside at a special pier either in bulk barges or in less-than-carload containers placed on deck barges. When bulk cargoes are unloaded the dust pump handles the dust directly in the barge; when container shipments are received it handles the material as it is unloaded from the containers.

The mineral dust is pumped through a 6-in., extra heavy, flanged steel pipe, extending from the pump to the silo, a distance of 450 lin ft (including an 85-ft lift). At the heads of the 3 silos a series of valves distributes the dust into the proper silo.

Each silo, about 18 ft in diameter, 30 ft high, and constructed of concrete block, has a capacity of 225 tons as pumped and about 275 tons after settlement. Each is provided with a bag dust collector at the point where the pumping air is exhausted.

In the less-than-carload container type of delivery, 20 to 21 containers are loaded onto a deck barge in such fashion that air and dust discharge hoses may be readily attached. Each container carries about 10 tons of dust. In this method, the discharge of the dust from the containers to the silos is accomplished in two successive steps. In the first step, the dust is transferred from the container to an auxiliary dust tank; in the second, it is fed from the tank to

[3a] 29 W. 39 St., New York 18, N. Y.

[4] "Specification for Asphalt Cement," *24-A-7-S*, Board of Standardization, Dept. of Purchase, City of New York, New York, N. Y., 1945.

[5] "Specification for Mineral Dust," *24-M-17-S*, Board of Standardization, Dept. of Purchase, City of New York, New York, N. Y., 1945.

a box enclosing the pump in a stationary position, from which the pump blows the dust to the silos. The function of the auxiliary tank is temporarily to collect the dust and control its flow to the pump.

Comparing the two methods under existing plant conditions, the container system of delivery was found to be the more efficient. In bulk deliveries quantities of from less than 400 tons to 500 tons per shipment are uneconomical. Inasmuch as the storage capacity at the plant is limited to 700 tons of dust as pumped and 800 tons of dust as settled, the reserve storage supplies must be drawn very low to provide space for these large shipments. Furthermore, the time lost in making displacement measurements, putting the pump into the barge, removing doors (which invariably jam), permitting dust to settle in silos, unloading the pump, and completing measurements adds up so rapidly that from 6 days to 8 days are required to unload a 500-ton barge. At full plant production the material would be used almost as fast as it was pumped, thereby leaving no reserves. The maintenance of sufficient reserve supplies is extremely difficult, not only at full plant capacity, but also at low storage reserve in the event of delayed shipments. With the container method—wherein shipments as low as 200 tons are economical, the quantity of shipment is accepted by certified weights on bills of lading, and the time lost by the various phases of the other method are eliminated—a shipment can be unloaded in 9 hours. Under such conditions the hazards of maintaining adequate reserves disappear, and even a time lapse for shipping delays may be allowed.

Again, in comparing the two methods, in the bulk barge method men must enter and work in the dust. This is far from satisfactory, and inexperienced operation of the pump can create great clouds of dust which are carried far and wide by the winds, creating a nuisance throughout the neighborhood. In the container method, the dust is almost fully enclosed from the containers to the silo, making this system almost dustless. From the standpoint of economy, under present plant conditions, the container method provides a material saving in labor costs by eliminating the cumbersome measurements and the handling of the pump and doors in the barge.

A third method under study provides complete pneumatic operation, having a larger capacity (70 tons per hr) and a longer effective pumping range (about 700 ft). Although this method can be used only with container shipments, the operating power costs compare favorably with both the other methods. In addition, it is extremely efficient, requiring only three men; it is completely dustless; its capacity is more than double the other methods; and, since it requires no moving parts nor electrical controls, it is almost ideal from the standpoint of maintenance and repair.

Manufacture of Paving Materials

The machinery within the plant is divided into two complete asphalt plant units, which may be operated singly or simultaneously. The units, each having an output capacity of about 350 tons per day, are designated as No. 2 and No.3. Space, foundations, and storage have been provided for a third or No. 1 unit in the event of future expansion. The units are alined side by side

in an east-west direction, this alinement extending through to the storage bins so that materials may be fed to either unit in the event of a breakdown.

The operation of the units in the manufacturing process may be divided into three integral steps, successive in operation but not continuous. In the first step the materials are handled and processed in passing from the bins to the mixers; in the second, the processed materials are proportioned and mixed; and in the third, the paving mixture is discharged into vehicles which convey it to the paving operations. In these three steps are included all the ramifications of operating, designing, and controlling the plant and a check test of the materials.

Processing of Materials

Transfer of Aggregates from Storage Bins to Driers.—The methods used in handling aggregates from the storage bins to the driers may vary with the special conditions of different plants. In this plant, where enclosed and protected bins are provided for the aggregates in order to assure year round operation, the system used is somewhat different from the usual plant installations.

In the storage-bin building, the bins for the sand and stone lie one behind the other, divided by a wall. Discharge openings are provided in the floor of the bins directly beneath the shuttle belts and over a tunnel alined with its corresponding unit in the plant. The openings are about 4 ft square and are fitted underneath with truncated pyramidal chutes and adjustable discharge gates. To assure steady flow of material through the gates, heavy-duty, portable vibrators are attached to the chutes. Experience shows that the stone moves freely through the gates without assistance, whereas the sand can be readily moved by the vibrators.

During the design stage, a drainage system for the bottom of the storage bins was studied. However, it was not adopted and its elimination has created several operating problems. Accumulated moisture drained from the materials, at first, ran through the discharge gates onto the belts, which in turn carried it into the rest of the machinery, causing corrosion and flooding. It was necessary to construct detachable scuppers on the gates to divert the flow from the belts to the tunnel gutters, which connect with a sewer. Although this method works well it is not wholly satisfactory because the tunnels are continually damp. Based on this experience, it is believed that all-enclosed bins should be provided with drainage systems to divert the drainage from the machinery.

The discharge gates feed directly upon belt conveyors operating in the tunnels. The conveyors, of the conventional rubber belt type with 18-in. belting and operating at a capacity of 80 tons per hr, are 170 ft in length and will carry sand or stone separately or together, depending on the requirements of the plant. The belts discharge into surge hoppers, each of about 5-cu-yd capacity, which act as equalizers between the belt feed and the feed to the elevators. Because uniform, continuous feed to the elevator is desirable, and because such flow cannot be secured in the operation of the belts, the surge hoppers regulate the flow by discharging into the elevator through a rheostat-controlled vibrating feeder. Furthermore, this system gives the drier operator finger-tip control of the material feed.

The material is raised about 40 ft from the surge hoppers to the drier feed by the cold elevator, which has a capacity of 75 tons per hr.

Driers.—The driers perform the function of drying and heating the sand and stone in preparation for their use in the paving mixtures. They are of the double-fired type, each having a capacity of 60 tons per hr. The drier drum is 28 ft in length, 6 ft in diameter, and is rotated in the usual manner at 8 rpm by a 40-hp constant speed motor. The drum is fired directly at the discharge end by a rotary oil burner operating in a cylindrical firebox 6 ft in length and 5 ft in diameter. The hot gases of this fire pass through the entire length of the drier. The middle of the drum is completely enclosed in a secondary rec-

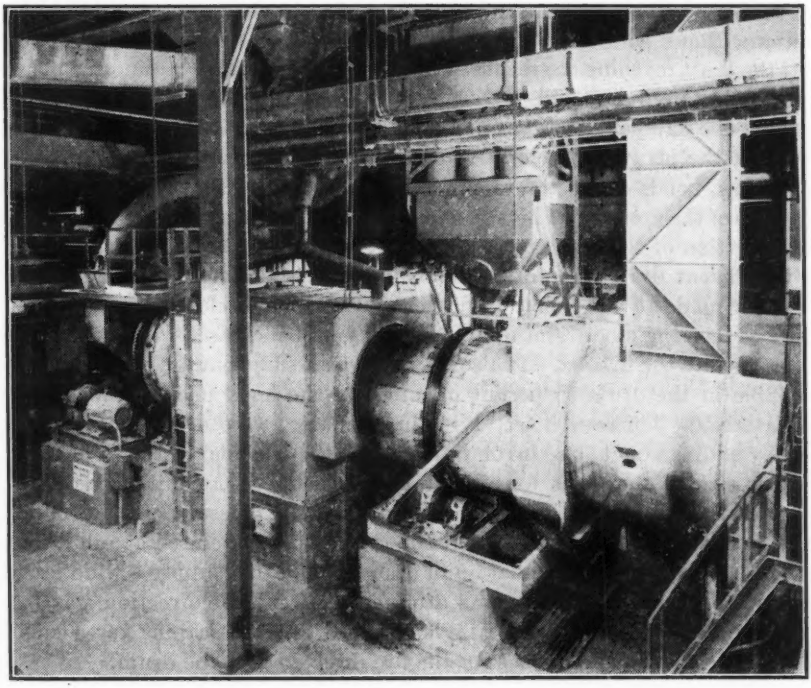

FIG. 4.—MATERIAL DRIER

tangular firebox, also fired by a rotary oil burner (Fig. 4). The hot gases of the secondary fire enter the interior of the rotating drum through 6 elbows fastened to the shell of the drum in the interior. The elbows face in the direction of the flow of the materials through the drier and are capped with hoods to prevent leakage of the material into the firebox. This permits entrance of the gases through the 2-in. space provided between the hood skirt and the elbow. Cascading vanes on the inside of the drum move the materials forward through the drum and at the same time drop them through the hot gas flow. The object of the secondary fire is to dry the materials at a point where the moisture may be removed quickly, thereby eliminating the zone of corrosion that appears in

driers of the single-fired type. The primary fire aids in drying, but functions predominately in heating the materials. By this combination, longer effective life for the drier and higher fuel economy are secured.

Acceptance of the driers was predicated on a test performance wherein the drier operating at a capacity of 60 tons per hr was required to process asphalt sand (New York City specification type 1) with a moisture content of 6% from a temperature of from 40° F to 400° F at a maximum consumption of 3 gal of No. 6 fuel oil per ton. These requirements were met in a series of tests on each drier. In normal operation, however, several problems developed. A heavy leakage of flue gases around the concentric rings of the secondary firebox in which the drum rotates caused extremely disagreeable conditions in the plant. Further, it was found that the secondary fire could not be operated at full power without considerable smoking. Analysis of the conditions indicated that the draft reaching the secondary fire was insufficient to maintain a negative pressure in the firebox and prevent escape of gases, or to draw in sufficient air through the dampers to provide full combustion of the oil. As it was imperative that the plant be properly ventilated in order to provide proper working conditions, hoods connected to the main draft of the drier were installed over the rings of the firebox. This expedient trapped the gases and slightly improved the operation of the fire; it also diverted some of the heat. It was apparent that sufficient draft was not available through the elbows, and adjustments had to be made at that point. Further examination disclosed that the annular bent between the elbow and its hood could not be increased without leakage of materials into the firebox. It also disclosed that sufficient draft velocities were available in the drum to permit openings in the hood itself without leakage of materials. Thirteen holes, 1 in. in diameter, were drilled in each hood, thereby adding about 50 sq in. of draft vent to the secondary fire. The number and the size of the holes were arbitrarily selected as an initial trial. With the added vent, results indicated that a negative pressure of $\frac{3}{8}$ in. of water was created in the firebox, and that the fire could be operated at full intensity without smoke. By manipulation of dampers and fuel supply a wide range of efficient control over the fire was attained. After the correction was made, the dampers on the concentric ring hoods used to trap escaping gases could be closed and the full effectiveness of the fire directed into the drum.

At the primary fire a very unusual condition was noted. In spite of the heavy draft through the drier, a back draft persisted in the firebox around the flame. Because of this back draft the sand that was being discharged from the drier was sucked into the firebox, where it settled, accumulated, and vitrified. Coatings of this vitrified sand as much as 6 in. thick accumulated in as little as 10 days of operation. To maintain the efficiency of the fire it was necessary that the coatings be removed periodically. Inasmuch as the coatings were so hard that small power chipping hammers were required, numerous cleanings were injurious to the firebox brickwork. In fact, relining of these fireboxes was necessary after 18 months of operation. Studies and tests give no results other than that sand with a high iron content builds up the coatings very rapidly.

The driers were equipped with automatic controls to coordinate the intensity of the fires with the desired temperature settings for processed materials under constant uniform feed conditions. Operation of the controls centers upon a recording pyrometer, at which the desired temperature of the processed materials may be set, and which records the temperature of the stack gases and the processed materials discharged. Through complicated electrical controls the relationship between stack and material temperatures is translated into the operation of the stack fans (through speed adjustments made by a fluid drive), the quantity of oil fed to the burners (by electrically actuated cocks), and the position of the fire dampers (on the rotary burners). Actually, it was found that the narrowest range in which the system could operate was within 50° F, plus or minus, of a desired temperature. Thus, instead of a steady temperature there was an unsatisfactory fluctuation of 100° F. Tests, adjustments, and changes failed to reduce this range to a suitable value. Studies disclosed that the variables of a wide range in moisture content and the combination of materials processed could not be handled on a uniform flow basis by the system. The time lag in correcting the fires was so great that overrun and underrun of the limits could not be avoided. As it requires 20 min for a particle of the material to pass through the drier, short period adjustments at the discharge end could not be made for temperature. Better results could be attained by varying the feed, but oil efficiencies began to decline.

Since the equipment was at hand, and since its operation required manual control of feed, it was decided to modify the system to one of simplified manual control. Fires are set at an intensity which, from experience, is relative to the average moisture content and the combination of materials, and also to the flow of the materials to maintain the temperature within desired limits. This conforms to plant operation wherein heavy flows are desired during a certain periodic output, and lighter flows, during replenishment of storage in hot bins in off periods.

Substantial fuel oil economies were secured by the modifications made in the drier and in its controls and by the operators as they become experienced. Skilled operators can maintain the discharge temperature of the materials within 25° F of the desired temperature. During full scale operation when 50 tons per hr of material are processed steadily for 5 hours or 6 hours, the fuel oil consumption has varied between 1.8 gal per ton and 2 gal per ton. Over a 1-year period, including winter months and rainy days when daily output was as little as 50 tons, the fuel oil consumption averaged about 2¼ gal per ton. Compared to specification requirements under steady operations at full output of a maximum of 3 gal per ton, a saving of 33% at full production and about 25% on over-all production has been realized.

After 18 months of operation the driers have performed well—with the exception of the points described, there has been slight warping due to overheating and slight buckling of the main support straps in the secondary firebox. The drum, rotating bands, girth and pinion gears, supporting rollers, and the interior shell, vanes, and elbows show very little wear.

Hot Elevator.—The hot elevator, which raises the heated processed materials from the drier discharge about 75 ft to a point of gravity feed to the screens,

is of the vertical continuous bucket type and has a capacity of 75 tons per hr. The elevator is driven by a 15-hp, constant speed, gear-reduced motor.

Difficulties were encountered almost immediately in the operation of the elevators. The conditions under which they operate are severe, in that they must handle, while still cold, materials at 400° F, and then, after reaching operating temperature, they are shut down at the end of the day to cool rapidly. Expansion and contraction in the chain linkage, due to the operating temperature conditions, amounts to about 2 in. at the take-up boxes. Operating the elevator with the take-ups set so that the chain was taut when cold created so much slack when the elevator warmed up that the buckets swayed, slapping the casing and hooking in the intake and discharge chutes. Operation with the chain taut when warmed up, and not released during cooling, bent and snapped the shaft. An intermediate setting at a point where slapping of the buckets and strain on the shaft were reduced and balanced proved unsatisfactory. Thus, it became apparent that adjustment during heating and cooling was necessary, and a counterweighted automatic take-up was therefore installed. This, however, required manual take-up on the screws to dampen the oscillations of the counterweight. Because continual attendance during the warm-up period was required in any case, it was decided to take up the slack only to a snug position, periodically, until maximum expansion was attained, and to release take-ups fully on shutdown. This procedure is not foolproof, but it has worked satisfactorily. Based on these experiences with the vertical type elevator, it is believed that a continuous bucket elevator of the inclined type, wherein slack is provided on the unloaded side of the chain, would eliminate the difficulties encountered.

Vibrating Screens.—The hot elevator discharges its processed material into the vibrating screens through a gravity chute sloping downward at an angle of 60° with the horizontal. The vibrating screens, situated atop the hot storage bins, separate the combined sand and stone and deposit each into its respective compartment of the hot bins. They also screen out oversize particles into a tailing chute. The vibrating screens used at this plant are of the horizontal type and have a capacity of 75 tons per hr. A motor drive imparts to the spring mounted screen carriage an elliptical motion ($\frac{3}{8}$ in. horizontally about the long axis and $\frac{1}{8}$ in. vertically about the short axis). The screens themselves are mounted in the virbating carriage in two layers, the upper layer being composed of a $\frac{1}{2}$-in. wire screen extending 8 ft ahead of a 4-ft bar screen with $2\frac{1}{2}$-in. openings. The lower layer is composed of a No. 4 screen extending the full 12-ft length. Between the two layers of screening and directly beneath the $2\frac{1}{2}$-in. bar screen a steel plate is introduced. Material entering the machine falls upon the $\frac{1}{2}$-in. screen, where the passing portion is dropped on the No. 4 screen and the retained portion is carried forward to the $2\frac{1}{2}$-in. screen. The portion passing the $2\frac{1}{2}$-in. screen falls upon the steel plate, which in turn deposits its charge into the stone compartment, the retained portion being carried forward into the tailing chute. Material passed by the $\frac{1}{2}$-in. screen onto the No. 4 screen is sifted into the sand compartment; that retained on the No. 4 screen is dropped into the stone compartment.

Although these vibrating screens operate quietly and efficiently, like the hot elevators they are subject to the same severe temperature changes, which similarly create trouble.

Hot Storage Bins.—Directly beneath and receiving the discharge from the vibrating screens are the hot storage bins, which in turn are directly over the mixers. The hot storage bins are compartmented into two equal sections, each of 50-ton capacity, for the storage of sand in both sections, or of sand and stone separately, in respective compartments. The hot storage bins are unique in that they are fully insulated by a 2-in. blanket and a ½-in. plastic insulation covered by a heavy canvas jacket.

The plant, in supplying paving materials to field repair crews, must furnish nearly 50% of its total daily output during the first few hours of its operation. Output then eases off until just before noon when the second peak load must be met. To meet the initial peak load effectively, the plant must either be started extremely early in the morning, or make provision for carrying suitable materials over from the preceding day. In the design of the plant the latter method was selected and the hot storage bins suitably insulated to meet this need.

Under normal operation the materials are delivered to these bins at temperatures between 400° F and 425° F. When the bins are being filled for overnight storage the temperature of the materials is raised to between 500° F and 550° F. In this manner, sufficient materials are made available for the production of more than 130 tons of paving materials the first thing on the following morning without even starting the driers. Nevertheless, the driers are started at the same time as the mixers, and within 20 min begin replenishing the bins. Thus, nearly 200 tons of paving materials may be produced by a single unit of the plant between 7:30 a. m. and 9:30 a. m. Because the production falls off after the initial surge, the driers which operate under constant load catch up and replenish the bins again ready for the second peak between 11 a. m. and 1 p. m., after which the temperature of the materials is raised and the bins are filled for the following day.

Utilizing self-reading pyrometers permanently installed in the hot bins extensive tests on the heat retention characteristics of materials stored in the bins indicate that fully loaded bins will lose from 50° F to 80° F in 18 hours during the summer, and from 100° F to 125° F in the winter. With the bins partly filled, temperature losses are considerably higher. Generally the loss in temperature is dependent upon the atmospheric temperature around the bin, and the quantity stored therein. On several occasions during the summer full bins were carried over from Thursday to Monday morning with the materials still warm enough for a satisfactory mix.

At times, long periods of inclement weather are encountered. On these occasions the materials in the bins fall below usuable temperatures, necessitating their removal from the bins. The bins are unloaded, and the material is either returned to the main storage bins or used for other purposes. An installation of auxiliary equipment that would return this material to the driers, thereby saving fuel and handling, is under study. However, correlation of bin filling activities with weather forecasts helps materially in avoiding this situa-

tion. The bins have functioned most satisfactorily and have met plant operational requirements ideally.

Limestone Dust.—Limestone dust stored in the silos is not processed in any way, and is fed directly (cold) to the mixers. Dust is drawn from the hopper bottoms of the silos through circular valves and conveyed to the mixers by rotary screw conveyors which are operated by motors controlled by push buttons at the mixer. Silo No. 3 feeds directly to mixer No. 3. Mixer No. 2 can draw dust from either silo No. 2 or silo No. 1.

Anyone who handles mineral fillers in bulk quickly becomes acquainted with their extremely contrary flow characteristics. Limestone dust when air activated flows like water, but when deaired and settled it packs so firmly that little short of brute force can budge it again. In silos 30 ft in height, like those in the plant, almost anything can happen—and usually does—while drawing the dust from them. The one essential, uniform flow in desired quantities, seldom occurs unless the silos are full. With the silo full, the dust flows smoothly through the conveyor. As the material is drawn out of the silo a crater of ever increasing depth forms, and things begin to happen. The crater continues to increase in depth until the angle of repose (as determined by existing conditions) is exceeded. Then the entire lip of the crater caves in, in a huge avalanche. The angle of repose is seldom the same as it varies with the atmospheric humidity, the amount of entrained air, the degree of compaction, the degree of absorbed moisture, the grain size, and other conditions. As neither the formation of the crater nor its estimated angle of repose may be observed during operation, no warning of immediate trouble is sounded. As the avalanche fills the crater the dust pours through closed valves, through stationary conveyors, and through every hole and crevice of the system out into the plant. At one time, 800 lb of dust poured into the weigh box of the mixer, through a closed discharge valve and 40 ft of stationary conveyor, on a day the plant did not operate. Another time, it passed the closed valve and poured into the conveyor, jamming the conveyor. By contrast, at times the avalanche will jam the dust at the pit of the crater and form an arch over the gate.

From these and other experiences it appears that uniform flow necessitates the elimination of the formation of craters which cause subsequent avalanches in the silos. Anticipating some difficulty, air jets were installed in each hopper leg of the silo during construction. The action of the jets was as unpredictable as the dust itself. Sometimes the high-pressure air would blow a small hole in the dust with no effect upon the formation of the crater. Then, after the crater formed, it would cause an avalanche. Low-pressure air was no better. Experience proved that single or double air jets were unsatisfactory. Multiple jets, as in the skirt of a blast furnace, were considered, but their adaptation was not feasible with the silos.

The problem of maintaining the dust level in the silos became a serious problem. In the spirit of "last hope" a light-duty electric vibrator mounted on the hopper of the silo was tried. Tests showed that constant operation of the vibrator packed the dust more solidly, whereas haphazard intermittent operation caused avalanching. However, interlocking of vibrator operation with the conveyor feed showed promise. Momentary operation just prior to

conveyor feed seemed to solve the problem, so a temporary installation of the vibrator was made with controls mounted at the mixer for extended operational tests. After 8 months of testing, during which satisfactory flow was maintained under all conditions, it was decided to make permanent vibrator installations.

Asphaltic Cement.—AC is stored in a composite storage tank of 125,000-gal capacity situated in the main plant building (Fig. 2). This tank is composed of five sections, all rectangular in shape, set in two tiers; two tanks in the lower, and three tanks in the upper tier. The two tanks in the lower tier, designated as tanks No. 1 and No. 2, each have a capacity of 30,000 gal. The three tanks in the upper tier, designated as tanks No. 3, No. 4, and No. 5, have a capacity of 20,000 gal, 25,000 gal, and 20,000 gal, respectively. The upper tanks, resting directly on the tops of the lower ones, are separated by two walkways—giving access to fill lines, inspection manholes, and vents required in the two lower tanks. Catwalks over the upper tanks serve a similar purpose. The entire composite tank and its upper sections are fully insulated by a 3-in. asbestos blanket and ½ in. of plastic insulating material covered by an 8-oz canvas jacket. The quantities of AC contained in the tanks are measured by a self-reading, multiple-liquid, level gage.

Each of the tanks in the composite tank was designed and is used for a specific purpose. Tanks No. 1, No. 2, and No. 3 are for main storage, wherein the AC is maintained at 270° F. Tanks No. 4 and No. 5 are used for service, in these the temperature of the AC is raised and maintained at 330° F.

A complete system of piping provides for the flow of AC from the barges to any one or all of the tanks; also, for the transfer of AC from one tank to another, or from either one or both service tanks to either surge tank, and from either surge tank to the mixers, together with returns from the surge tanks. Two motor driven asphalt pumps, each having a capacity of 120 gal per min, may be used in transferring AC within the composite tank or from service tanks to surge tanks. Two air compressors transfer the AC at relatively uniform pressure from the surge tank to the mixers.

All the storage tanks conveying pipe lines, transfer pumps, and meters handling AC, as well as the paving material mixers, are electrically heated—thereby completely replacing the conventional method of high-pressure steam for this purpose. Although this method of heating is not original with this plant, it is, however, the first complete large scale installation ever made for this purpose in an asphalt plant. Little precedent was available either for design, construction, or operation of such an installation.

"Calrod" heating elements of varying lengths and heating intensities are utilized for the heating of piping, tanks, mixers, other equipment, and appurtenances. The heating of AC in tanks is automatically controlled by recording thermometers operating in conjunction with time switches. The heating of pipes, mixers, and other equipment, either directly by the elements, or indirectly by the circulation of heat transfer oils heated by immersion "calrods," is operated manually by push-button switches with temperature control maintained by thermostats. The system has operated effectively and efficiently

after its operational characteristics were determined and an operational procedure developed.[6,7]

Mixers

Because of the peak demands on the plant and the uniform and continuous operation of its processing machinery, it is evident that the mixer, which is the heart of the production equipment, must have a huge capacity if it is to meet the high intermittent demands.

In the design phase some controversy developed concerning the feasibility of large capacity mixers for the manufacture of high-grade sheet asphalt paving mixers. The alternative to employing large mixers was the installation of batteries of smaller mixers operating simultaneously in order to meet the time requirements of the production output. Inasmuch as space was at a premium, and simultaneous operation of dual mixers posed severe complications in operating equipment and necessitated an increase in operating personnel, it was decided to use large 4-ton mixers.

Each mixer of the usual pugmill type has a capacity of 8,000 lb per batch. Two rotary shafts, each fitted with 28 blades and rotating in opposite directions, knead the materials downward against the mixer liners. The blades are fitted with replaceable and adjustable tips, 8 in. square. An 125-hp, 3-phase, 1,150-rpm electric motor drives the shafts through a chain gear drive and reducing gear. Shaft speeds can be adjusted in steps of 5 rpm from 45 rpm to 60 rpm by gear changes in the chain drive. The mixer body is lined with interchangeable, reversible, and replaceable cast manganese steel liners $\frac{3}{4}$ in. thick. All shaft bearings are of the heavy-duty, self-alining, spherical, roller-bearing type. The gate under the discharge opening in the body of the mixer is actuated by a double-acting air cylinder of special design, operated by air pressure at 100 lb per sq in.

Above the body of the mixer, and directly below the discharge chutes of the hot bin, a weigh box is suspended having a capacity of 8,000 lb of materials. The scale of this weigh box has 10-lb dial graduations to a capacity of 10,000 lb. The gate used in discharging the contents of the weigh box is actuated by a small air ram operating on compressed air at 100 lb per sq in.

Injection of AC into the mixer is accomplished by a spray bar of unusual trapezoidal cross section situated above the mixer blades and extending across the front side of the mixer. This spray bar is fitted with suitable baffles to assure uniform distribution of AC through 16 jets, uniformly spaced and adjustable in size from spray tip to $1\frac{1}{4}$-in. diameter.

The entire mixer—composed of the body, blades, weigh box, hot bin discharge chutes, limestone dust conveyor chute, and spray bar—is entirely enclosed in a dustproof housing. Air lines, electrical conduits, and levers for the actuation of the discharge gates of the hot bins all pass through sealing gaskets in the housing.

The AC meter, limestone dust feed control, and interlocking solenoids are situated on a bench formed by the body and the housing at the front of the

[6] "Electric Heating for Asphalt Plants," by Ladis H. Csanyi, *Engineering News-Record*, April 1, 1948, p. 82.

[7] "Asphalt Plant Operating Costs," by Ladis H. Csanyi, *ibid.*, April 15, 1948, p. 90.

mixer. The AC is measured and injected into the spray bar by a meter having a capacity of not less than 100 gal of AC in 30 sec, and operating at a pressure of 30 lb per sq in. Because the accuracy of the meter depends largely on the temperature of the AC, it is essential that this temperature be maintained between the operating limits of 320° F and 330° F.

At this point a discussion of the surge tank is essential to an understanding of its function. The surge tank serves a dual purpose. First, since the AC pumps have a capacity of 120 gal per min, it is evident that they could not furnish sufficient AC to meet the meter requirements of not less than 100 gal in 30 sec, or 200 gal per min. The surge tank balances this discrepancy by storing sufficient AC during the mixing cycle to satisfy the meter requirements. Inasmuch as the pump operates continually during the 90-sec cycle between AC discharges, there is ample capacity to recharge the surge tank. Furthermore, the surge tank is of sufficient size to meet the double load caused by the simultaneous demand of both mixers. Secondly, for operation the AC meter requires an AC fluid pressure of 30 lb per sq in. In filling the surge tank in the morning, an air pocket is trapped at its top. By using pressure regulating valves on the AC return circulating lines, the pressure of the air pocket may be retained at the required 35 lb per sq in. Thus, by the utilization of the surge tank in the AC feed line, smaller pumps are adequate for pumping AC, and a simple method of maintaining the necessary fluid pressure is provided.

The mixer body, spray bar, and AC meter are all either directly or indirectly heated by electricity. Originally, the mixer body and spray bar were heated by electric strip heaters. Because of the high temperatures developed during the heating, the special heat-resistant insulation on the wiring to these heaters failed. Therefore, the strip heaters were removed and replaced with "calrods." In a "calrod" installation all wiring is outside the heated zone, thereby eliminating the need for special wiring. By this modification more rapid and more uniform heating resulted, the heating time being reduced from 5 hours to $2\frac{1}{2}$ hours. The meter and its screen were originally heated by the circulation of a heat transfer oil, electrically heated by an immersion "calrod". This method was unsatisfactory because the heating time required was from 7 hours to 8 hours and a heavy overload developed on the heat transfer oil circulating pump. Examination of this system disclosed that the oil lines in the head of the meter were crimped in many places. Replacement without crimping was impossible, due to the tortuous coils and twists, so the piping was abandoned and "calrods" were installed around the head of the meter and its screen, leaving only the body of the meter to be heated by circulating oil. By this change the overload on the circulating pump was eliminated, and the heating time was reduced from 7 hours to 2 hours.

Semi-automatic operation was intended by the design of the mixer. The weighing of materials was to be performed manually and then, by a system of interlocked controls, the mixing and injection of AC was to be performed automatically by the pressing of a button. Manual discharge of the mixer was to be restricted until the mixing cycle was completed. The system worked, but an analysis disclosed that it was not foolproof. By virtue of its operation, the meter required a resetting after each measurement of AC. If this resetting was

overlooked, the next measurement of the meter would be 10,000 gal of AC. Hence, rather than burden the mixer operator with the responsibility of avoiding a flood of searing hot AC, it was decided to modify the system and restrict its function to a safety measure. In this alteration the interlocking device prevents tripping of the weigh box discharge gate before the mixer is emptied and the gate closed, and aids the operator in timing the mixing cycle.

In operating the mixer, the operator weighs the proper quantities of materials into the weigh box. After emptying the contents of the weigh box into the mixer a 20-sec dry mix is made. Injection of AC follows after which a 40-sec wet mix period is provided. The 4 tons of paving mixture are then discharged into a truck below the mixer gate. Signal lights, "klaxons," and a peep hole assure the operator that he is discharging his mixer into a truck and not into the truck runway. Thus, the mixing cycle requires from 100 sec to 120 sec. Each mixer produces 4 tons of paving materials every 2 min, or 120 tons per hr. To assure full efficiency the mixers are offset, one over each of two truck lanes in the runway. Hence, with both mixers operating, the plant can produce at least 240 tons per hr while the hot bin supplies last.

During an 18-month period, with the exception of the few minor difficulties, the mixers have operated most satisfactorily. The character of the product indicates that the mixers consistently produce good, uniform mixes. The wear on the blade tips, liner plates, and discharge gates has been negligible. The spray bar jets were varied from spray tips up to full $1\frac{1}{4}$-in. openings during the period. Orifices smaller than the full opening tended to clog with carbonized AC and were gradually increased in size until full openings are in use with little or no effect upon the uniformity of the mix. As for shaft speed, 45 rpm is the nearest speed available to provide best results, but it is believed that 42 rpm would eliminate the slight splashing of the material noticed at 45 rpm. Based on the knowledge gleaned through experience and the results obtained with these mixers, it appears that the selection of large mixers was a good one and any trepidation concerning their operation and their ability to produce good, uniform mixes has been completely dispelled.

Dust Control

The control of dust expelled from such a plant and escaping from the machinery therein has received increasing attention during the past decade. The extent and the intensity of the interest generated may be traced to the degree of the nuisance committed on the neighborhood, the increasing realization of the cost of dust waste, and the appreciation of the detrimental effects of the dust on operating machinery and on the working conditions of the plant personnel.

As asphalt plants either entered more populated areas, or were hemmed in more closely by other industries, the great clouds of yellowish brown dust billowing from their stacks became more and more objectionable. In some cases the condition was so bad that the plant was confronted with the alternatives of moving or of providing some method of abating the nuisance if harmonious neighborly relations were to be preserved. Increasing costs of materials further emphasized the undesirability of dust clouds by stressing the

costly wastefulness of usable materials. With the development of this new conception of dust, fugitive dust within the plant also gained new importance. Increasing costs and difficulty in securing new machinery or parts prematurely worn out by the abrasiveness of the dust, and the loss of trained men to other industries where working conditions were more pleasant and agreeable all aided in crystallizing the appreciation of the true problem of free dust.

At the plant in Manhattan, situated as it is in a high class residential area, the significance of complete dust control was thoroughly appreciated. The design and the operation of the plant were and are consequently directed to the attainment of the highest feasible degree of dust control. The method of control adopted was of the two-stage type, wherein heavy coarse particles are removed in the primary stage and the light fine particles in the secondary phase of the system. The phases of the process and the entire process are parts of the main blower exhaust system of the plant.

Each unit in the plant possesses its individual and separate dust control and main blower exhaust system and each of the latter includes a main duct exhaust fan having a capacity of 30,000 cu ft per min, driven by a 60-hp motor operating through an adjustable speed fluid and a V-belt drive to yield a maximum fan speed of 980 rpm. The exhaust fans have rectangular intake ducts 69 in. by 20 in., and a circular exhaust stack 42 in. in diameter. Under operating conditions of 60 hp, fan speeds of from 860 rpm to 920 rpm are developed; the capacity ranges from 24,000 cu ft per min to 28,000 cu ft per min at a static head of 7 in. of water at the fan intake. Input power and air capacity vary within limits with atmospheric conditions, stack temperature, and moisture content of the materials processed.

The main exhaust fan performs a dual purpose by acting as the main draft for the drier fires and as a secondary draft for the collection and conveyance of the dust from the various machines. A main draft duct 36 in. in diameter conducts flue gases and dust from the drier to a "multiclone" precipitator. Secondary ducts provided with adjustable dampers convey dust from the vibrating screen and the mixer through lines 12 in. in diameter to the hot elevator. From the hot elevator a line 24 in. in diameter conveys the accumulated dust to the precipitator. All these machines are built as dust-tight as possible and operate under negative pressure. The multiclone discharge is connected to the fan intake by a rectangular duct about 10 sq ft in cross section. The dust precipitated in the multiclone is discharged into the hot elevator intake chute by a motor driven rotary valve for re-use in the plant.

At first some difficulty was encountered in securing the desired results from the system, but complete control of fugitive dust in the plant was attained by damper settings whereby about 24,000 cu ft per min of the total intake fan capacity of 28,000 cu ft per min is allocated to the drier exhaust and about 4,000 cu ft per min to fugitive dust control. When the plant processes material having an average moisture content of 6% at the rate of from 45 tons per hr to 50 tons per hr in the drier, dust samplings taken at the intake and discharge of the multiclone indicate a removal of from 80% to 85% of the total dust entering the precipitator. The dust sampling of the stack discharge indicates a dust content of 2.25 g per cu ft of air.

Only the primary phase of the dust control system—namely, the multiclone precipitators—has been installed. Contracts for the installation of the secondary phase (electrical precipitators) are in progress. It is expected that the secondary phase will remove 90% or more of the discharge, giving the entire system an over-all efficiency of about 98%. The air and gases exhausted from the system then will be purer than the atmospheric air as far as dust is concerned.

Operation of the system as installed, although smooth and efficient as far as it goes, is unsatisfactory because the residual dust from the multiclones is being exhausted from the stack. The quantity, although far less than in most other plants, is still objectionable, indicating that a primary system in itself is insufficient and must be augmented by a secondary system to yield satisfactory dust control. Prevailing winds blowing the dust out over the East River have so far favored the plant, and the objectionable dust has as yet created no great degree of nuisance.

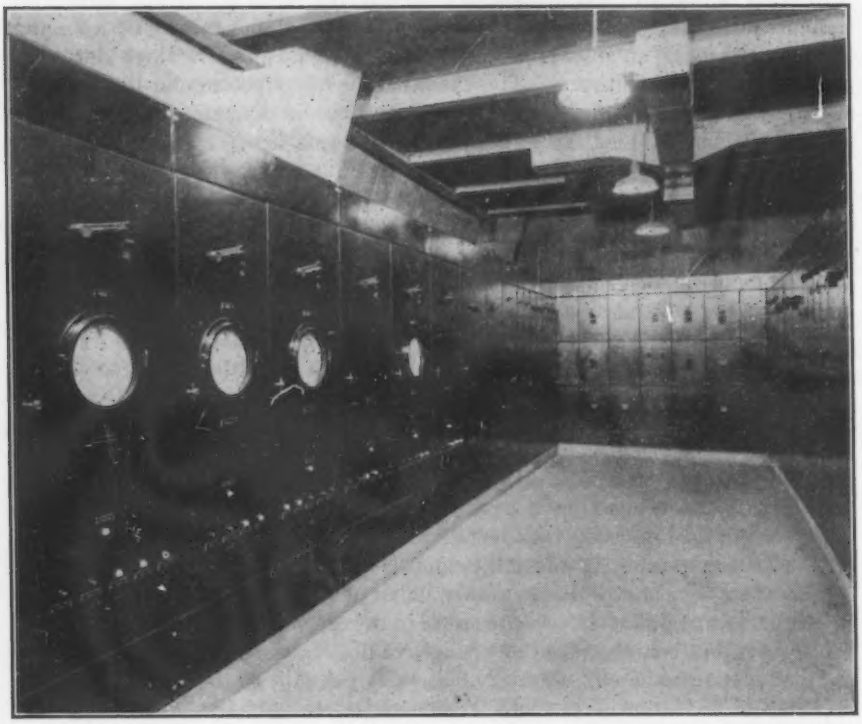

FIG. 5.—MAIN ELECTRICAL CONTROL ROOM, SHOWING ASPHALT TANK RECORDING THERMOMETERS

ELECTRICAL CONTROLS

Electrical power used by the plant, repair shops, and other facilities in the plant area is supplied at the rates of $1.75 per kw for demand and $\frac{3}{4}$¢ per kw-hr for consumption. A special building contains the necessary equipment to

transform the power in the main supply feeders to the requirements of the plant. This equipment is owned and serviced by the utility company. In another section of this building are located the plant's main power circuit breakers, which are owned, operated, and serviced by New York City. Power furnished to the plant is 208-volt, three-phase, 60-cycle current for power, and 120–208 volt, three-phase, 60-cycle current for lighting.

In the control room (Fig. 5) the power is distributed to the various machines through a U-shaped control panel board. Separate panels of this board contain all the necessary electrical devices and controls for each piece of machinery—such as the driers, mixers, exhaust fans, etc.—for simplicity in tracing breaks and in electrical maintenance and repair. Controls for the electrical heating of each storage or service AC tank similarly are contained in separate panels. From the panels the wiring runs to the operating control stations for various machines. In most cases, operating control stations contain the controls for several pieces of machinery whose operations are interrelated or must be started or stopped in proper sequence. In this manner all conveyor belts from dock to bins are operated from one control station, and the heating of all lines, pumps, and other appurtenances is controlled from another. Similarly a single control station operates the reclaiming belt, cold elevator, drier, hot elevator, vibrating screen, fuel oil pumps, oil burners, and exhaust fan of a single plant unit. A similar station controls the other unit, and also contains the controls for the building ventilator fans. Because these machines must be started or stopped in a definite sequence, the controls are mounted in the station in that order. With controls so organized, operating interlocks and personnel safety controls can be readily provided.

Costs

Considerable interest has been displayed in the operating costs of the various phases of production and in the final cost of materials manufactured by this plant. The elegance of the buildings, their location, the intricate controls, the extraordinary dust control, and the other seeming extravagances create grave doubts concerning the ability of the plant to produce paving materials at competitive or even reasonable costs. Visitors are amazed when informed that the plant can manufacture its products economically and even competitively. When apprized of the actual cost of the plant ($1,000,000 exclusive of land value), the perplexity increases, or the certainty of some subterfuge becomes even stronger. To commercial operators, the carrying charges on such an initial investment alone would be staggering, without adding the labor costs of what appears to be a comparatively large personnel. Can a visitor be blamed, after a superficial inspection, for wondering what form of mental gymnastics perpetrates the bland statement that costs are reasonable?

In comparing costs, it must be remembered that costs are relative and should be collated on a basis of real value received. To a city responsible for public safety and convenience, over-all costs and availability of the materials when required are of greater importance than specific costs. A contractor operates an asphalt plant during the paving season at high volume and low

specific cost. When volume falls off and specific cost rises above an economical limit he shuts down and lays off his force. A city cannot do this. Public safety and convenience must prevail at all times, and repairs must be made in or out of season. Under such conditions a ton of asphaltic paving material may be worth a great deal, particularly when all the manufacturers of this commodity are hibernating. Again, a contractor builds a plant with an eye to as great a return on investment in as short a time as possible for, by the vicissitudes and competitive nature of the industry, he may be out of business in a year. The city, on the other hand, will remain in business as long as streets and roads exist and require repairs, and can therefore project its returns on investment over a long period of time. Furthermore, the city's borrowings are tax exempt and consequently its bonds are attractive to investors at very low interest rates. In addition, the city sees no need for taxing its own property. It will be seen, therefore, that many factors enter into the costs to make them relative, and dependent almost solely on the value received by the purchaser.

The city, acting in the dual capacity of manufacturer and purchaser of asphaltic materials produced in a city owned plant, is interested in specific costs only in so far as they affect over-all costs. The specific costs of manufacturing a ton of sheet asphalt, for instance, are submerged in the over-all cost of placing this material in a pavement, regardless of season or weather, to maintain public safety and enhance public convenience. Nevertheless, it is interesting to examine the specific costs involved in the production of asphaltic materials at the plant.

Specific costs, again, are relative, for they depend greatly on the relationship between plant capacity and plant output. In a plant owned and operated by the city this relationship will vary from very low output to full capacity from day to day, depending on weather conditions the whole year round. Of course, on days when emergency paving is impossible the plant will not operate. Thus, specific costs will vary between exhorbitant values and very economical values. The extreme costs are not comparable with commercial plants that operate only when the demand in relation to capacity is sufficiently high to assure a return above overhead, operating costs, and material costs. Therefore, the most suitable basis for comparison of tonnage cost may be derived by dividing actual annual or monthly costs by the average daily output and by the average number of operating days per year or month, and then adding the actual cost of materials.

Before applying the formula, the average daily output and the average number of operating days must be ascertained. Although the plant has a maximum average output of 375 tons per day per unit, this quantity is not applicable because actual output may vary from 450 tons per day per unit during the height of the paving season to 50 tons per day during the winter in emergencies. Experience has shown that an average daily output of 300 tons per day per unit can be attained over the operating period A survey of 30 years of operating experience at the old plant discloses that the average operating time prevalent in the New York City area is 212 days per yr. This annual average may be broken down to 21 days per month for 10 months, thereby

permitting 1 month for vacations in the summer and 1 month for plant overhaul in the winter.

In applying the formula it is best to reduce separately each factor involved in the costs—such as labor, power, materials, and overhead. By this method each phase may be compared individually, and the whole collectively.

Labor.—Labor at the plant is paid on an annual basis, so that the actual annual labor cost is used. This cost includes full time for supervisory, operating, maintenance, and repair forces comprising the full plant complement of twenty-one men. The total annual payroll, during 1945, for these men was approximately $55,000. By applying the formula and the factors previously discussed, the cost of labor is found to be $260 per operating day, or 87¢ per ton.

Power.—Power is paid for on a monthly basis; therefore, actual average power charges per month are used. This cost includes the operation of one unit, all the material conveyor belts, the dust pump, heating of AC tanks and lines to both units, all other equipment required in operation, and light. The average monthly charge for power is approximately $1,300. By applying the formula and the factors previously determined, the power cost is found to be $62 per operating day, or 21¢ per ton.

TABLE 1.—Costs of Materials, in Dollars, for Various Plant Products (1945 Prices)

Product	Ingredients					Total (per ton)
	Sand (per cu yd)	Mineral dust (per ton)[a]	AC (per gal)	Stone (per cu yd)	Fuel Oil No. 6 (per gal)	
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Sheet asphalt (top)	0.92	4.50	0.0611	....	0.0439	2.88
Binder	0.92	....	0.0611	1.55	0.0439	1.85
Asphaltic concrete	0.92	4.50	0.0611	1.45	0.0439	2.21

[a] In bulk.

Materials.—The cost of materials (Table 1) is based on 1945 prices; the cost of fuel oil is predicated on the maximum use of 3 gal per ton of product.

Overhead.—It is in the calculation of overhead costs that variations occur. To avoid any stigma of subterfuge, an outline of the costs involved is given (Table 2) so that the reader may combine them in any way he pleases and determine his own results.

The outline indicates a maximum overhead cost of 80¢ per ton. Actual expenses, however, consist of only the interest on capital investment, and repair parts, lubricants, etc.—or a total of 33¢ per ton. As time goes on, the replacement of an entire piece of equipment may be necessary, involving some part of the 31¢ set aside for depreciation of machinery. As a result of preventive maintenance this may not be likely, but it is reasonable to assume that preventive maintenance will double the effective life of the equipment and extend its depreciation to 40 years. By this time some of the equipment will

be obsolete, but by proper replacement and gradual modernization it can be maintained in service. Thus, depreciation charges on the machinery may be reduced to 16¢ per ton. Depreciation charges on the structures are not included, for by their character they can far outlast their usefulness. The remaining $100,000 of the original $1,000,000 cost is allocated to the electrical dust precipitators not yet installed. After this installation depreciation on machinery will be based on $500,000 instead of the present $400,000. By this

TABLE 2.—OVERHEAD COSTS, IN DOLLARS

Description	Principal	Time (Yr)	Costs	
			Annual	Per ton
(1)	(2)	(3)	(4)	(5)
Interest on capital investment[a]	900,000	....	18,000	0.28
Depreciation on structures	500,000	50	10,000	0.16
Depreciation on machinery	400,000	20	20,000	0.31
Repair parts, lubrication, etc.	....	....	3,000	0.05
Total	....	....	51,000	0.80

[a] At 2%.

calculation a reasonable overhead of 49¢ per ton is obtained. Adding this overhead cost to that of labor, power, and materials gives the over-all costs as shown in Table 3. These costs compare favorably with prices quoted by contractors in the area for similar materials.

TABLE 3.—TOTAL COSTS, IN DOLLARS PER TON

Description	Sheet asphalt	Binder	Asphaltic concrete
(1)	(2)	(3)	(4)
Labor	0.87	0.87	0.87
Power	0.21	0.21	0.21
Materials	2.88	1.85	2.21
Overhead	0.49	0.49	0.49
Total	4.45	3.42	3.78

The preceding costs were based on the operation of only one plant unit. When labor conditions improve, and field paving forces are recruited to full strength, and when field requirements increase to 600 tons per day, operation of both units will be required. Under these conditions the costs on a tonnage basis will decrease materially. Only three additional men will be required to operate the second unit, therefore annual labor costs will only increase $7,500 to a total of $63,000. Applying the formula, labor costs decrease to 50¢ per ton. Power costs will also decrease, because the demand costs of material handling will be but one half of their present ratio to operating demand costs, to 16¢ per ton on an average of 600 tons per day. Material costs will obviously

remain constant, and overhead costs will decrease to one half of their present costs on fixed charges. On repair parts and lubricants they are expected to double, thus giving an estimated overhead cost of about 32¢ per ton. Applying these savings and compiling the costs on a basis of 600 tons per day gives $3.86 per ton for sheet asphalt, $2.83 per ton for binder, and $3.19 per ton for asphaltic concrete. These total costs of manufacturing a ton of the respective materials not only compare favorably with prices quoted by private contractors, but also are definitely competitive and produce a substantial annual saving to the city in the maintenance and repair of its streets. It is emphasized again that the plant produces materials for maintenance, repair, and resurfacing of streets and highways in the Borough of Manhattan only, and not for new construction or complete repaving of streets, which is done by private contractors.

Conclusions

The Municipal Asphalt Plant of the Borough of Manhattan, after two years of operation, has clearly demonstrated that an asphalt plant, properly equipped and suitably housed, can enter the heart of an urban area and be fully accepted as a satisfactory neighbor. By this development a multiple-unit plant can displace the usual practice of utilizing numerous small plants scattered about the outskirts of a city. The concentration of production at a focal point yields not only higher efficiency at reduced cost at the plant, but also greater efficiency at reduced cost in the transportation of materials from the plant to the field.

This plant has firmly established the economy and efficiency of electrical heating of AC, with all lines and equipment appurtaining thereto, for large stationary plants. By the innovation of insulated hot bins it has developed a new operational procedure which lends far greater flexibility to plant production. It has also definitely demonstrated the feasibility of large 4-ton batch mixers in producing high-grade paving materials. To the files of experience it has added a profusion of data concerning the operation and control of modern plant equipment and accessories. Lastly, it is hoped that by virtue of this treatise it may assist other plant designers and operators in utilizing some of the information gained.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

DULUTH-SUPERIOR HARBOR

BY HESTON R. COLE,[1] ESQ.

SYNOPSIS

This is a history of the development of the harbor at Duluth, Minn., and Superior, Wis., into the most important bulk traffic port in the world. There is outlined the growth in traffic from the days of the early French explorer and fur trader to the present, with weekly shipments of more than 1,000,000 tons of iron ore.

Construction of the harbor by the Corps of Engineers, Department of the Army, began in the days when there were still many wooden sailing vessels, which were loaded and unloaded by wheelbarrow and plank. Today, the harbor is being maintained and improved to accommodate the fast, deep draft, 600 ft long, steel vessels with enormous cargoes, which are loaded and unloaded at docks designed to accomplish this work quickly and efficiently by the most modern mechanically operated equipment.

Duluth-Superior Harbor is one of the finest natural harbors and probably the most important bulk traffic port in the world. Herein is sketched briefly its history, the types and tonnages of traffic, and a description of the harbor before and after improvements (see Fig. 1).

The first white men to visit the western end of Lake Superior were French explorers and traders, in 1659 and 1660. They left no written record of visiting Superior and Duluth bays, but there is reliable evidence that they wintered at Chequamegon Bay, 75 miles east of Duluth, and theirs was the first shipment (furs) from Lake Superior. Jesuit cartographers in Paris, France, drew the first map of Lake Superior in 1670–1671 and marked thereon a river flowing into Lake Superior at its western extremity, probably from information furnished by missionaries.

It is not until 1679 that there is written record of a lasting settlement at Head of the Lakes. During this year Daniel Greysolon Sier Du Lhut and his party portaged over Minnesota Point near its base, entered the St. Louis River

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by April 1, 1949.

[1] Col., Corps of Engrs., Dept. of the Army, Duluth, Minn.

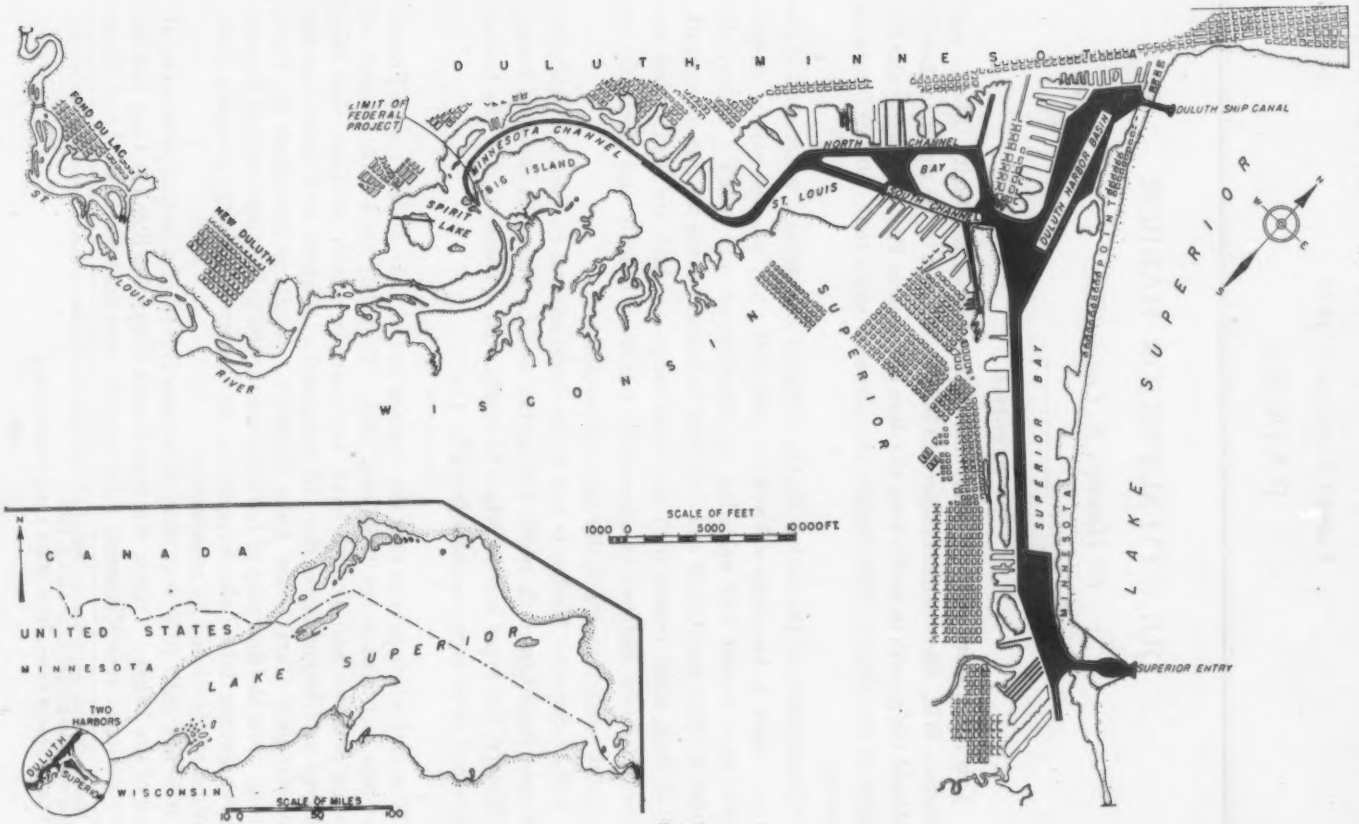
D U L U T H, M I N N E S O T A
LIMIT OF FEDERAL PROJECT
POND DU LAC
ST. LOUIS RIVER
NEW DULUTH
SPIRIT LAKE
BIG ISLAND
MINNESOTA CHANNEL
NORTH CHANNEL
BAY
ST LOUIS
SOUTH CHANNEL
DULUTH HARBOR BASIN
DULUTH SHIP CANAL
W I S C O N S I N
SUPERIOR BAY
MINNESOTA POINT
LAKE SUPERIOR
SUPERIOR ENTRY
SCALE OF FEET
1000 0 5000 10000 FT.
CANADA
UNITED STATES
MINNESOTA
LAKE SUPERIOR
TWO HARBORS
DULUTH
SUPERIOR
WISCONSIN
SCALE OF MILES
0 50 100

FIG. 1

and established a fur trading post 20 miles west of Duluth, which he named Fond du Lac, or end of lake. In 1856 the struggling townsite on the north side of Superior Bay adopted the name of Duluth in honor of this French explorer. The fur trade continued as the main commerce until 1855, the year the first lock was constructed at Sault Ste. Marie (Mich.).

The modern story of Duluth-Superior Harbor as a port begins in 1852 with the arrival of George Stuntz. Mr. Stuntz had been in charge of a survey party running the Wisconsin-Minnesota boundary line and liked the area so well that he settled on the spit of land later called Minnesota Point. Mr. Stuntz prospected over the area and, in 1865, found iron deposits while looking for gold. In 1873 he persuaded Philadelphia (Pa.) capital to invest in mining operations on the eastern end of the iron range. This was the beginning of the iron ore period. Although this ore was shipped from Two Harbors (Minn.), Duluth became the administrative headquarters for mining operations in northern Minnesota. Discovery of ore at Mountain Iron (Minn.), in November, 1890, by the Merritt brothers, and subsequent development of the Mesabi Range with the ever increasing shipments of ore through Duluth-Superior Harbor, soon made the port one of the most important in the world.

Duluth-Superior Harbor consists of an outer and inner basin. The outer basin, known as Superior Bay, lies between the mainland and the natural sand and gravel breakwater, which is ¼ mile wide and about 9 miles long and extends from the Minnesota side to the Wisconsin shore. This spit of land protects the harbor from disturbing wave action in Lake Superior. The inner harbor consists of St. Louis Bay and St. Louis River. Originally, there was only one entrance to the harbor, located at approximately the present position of Superior Entry near the south end of Superior Bay. The natural entrance shifted considerably from year to year, because of wave action and currents, and was often obstructed by shifting sand bars.

Superior Bay (the main harbor) and St. Louis Bay (the inner harbor) were wide expanses of shallow water with depths ranging from 5 ft to 9 ft and containing several small islands. The natural channel, created by the river current, was from 100 ft to 500 ft in width with depths varying from 8 ft to 12 ft. Because of the varying depths of water at the entrance to the harbor, navigation was limited generally to 8 ft or less, and during bad northeast storms this was decreased to 3 ft or 4 ft. These data are taken from a survey of the harbor made in 1861 by then Capt. George B. Meade, Corps of Engineers, who later attained fame as the victor at the Battle of Gettysburg.

The first act performed by the federal government was the building of a lighthouse at the entrance of the harbor in 1857. Parallel timber crib piers were constructed at Superior Entry by the Corps of Engineers from 1867 to 1875.

The present Superior Entry piers and breakwaters were completed in 1914. The piers are 500 ft apart; each has a concrete superstructure, supported by timber piling. The south pier is 1,584 ft long and the north pier, 2,096 ft long. The rubble mound breakwaters converge, forming an arrowhead and providing a stilling basin of approximately 62 acres, which gives safe passage in the

stormiest weather to all vessels. Entering from the lake at Superior Entry, the channel depth is 32 ft, decreasing to 25 ft at the Superior Bay end.

A channel between these piers which extends to established docks was dug to a project depth of 12 ft, the work being completed in 1873. This depth was the controlling depth until completion of the 16-ft project depth in 1885. Commerce that year totaled 1,200,000 tons valued at $24,500,000. Arrivals and departures totaled 1,997 vessels, of which approximately 10% were sailing ships. The largest cargo was about 3,000 tons. In 1885 there were 25 docks in the harbor and the elevators had a storage capacity of about 9,000,000 bu of grain. About 600,000 tons of coal were received and 14,000,000 bu of grain shipped.

With the opening of the Canadian and Poe locks at the Soo canals in 1895 and 1896, respectively, and with the advent of larger bulk freighters with steel hulls, the channels in the Duluth-Superior Harbor were deepened to 20 ft, with additional depths provided in the lake approach channels. During the six-year period, from 1896 to 1902, about 21,500,000 cu yd of material were dredged at a cost of about $2,000,000. Contracts were let at the low rates of 7½¢ per cu yd, 8¢ per cu yd, and 10¢ per cu yd, depending on the location. The first shipment of iron ore from Duluth-Superior Harbor was made in October, 1892, and continued shipments soon established the harbor as one of the most important on the Great Lakes. The billionth ton of iron ore was shipped in 1940 and more than 2,000,000,000 tons of total commerce were handled prior to July, 1947.

During the years from 1933 to 1935 the harbor was dredged to provide channel depths of 25 ft to the ore docks and 22 ft to the grain and coal docks. These depths were required to permit loading of the bulk freighters to their maximum usable draft.

In 1884 the bottom of the harbor was somewhat irregular, and vessels experienced a great deal of trouble from striking shoals. In 1893 some of the leading citizens of Superior and Duluth decided to cooperate in their efforts to secure harbor appropriations instead of fighting each other's requests for harbor development. The Joint Duluth-Superior River and Harbor Commission was created. Largely through the efforts of this commission an act was passed by Congress creating a special committee to examine and report on the harbor. In May, 1895, this examination of the harbor was made by a congressional river and harbor committee. While making the examination, the boat conveying the party passed a large freight boat that had accidentally gone aground in the Superior Front Channel. Later in the day, the inspection boat also ran aground and had to be pulled off by a tug. The committee decided the channel should be deepened. The separate harbors of Duluth and Superior were united under the title, "Harbor at Duluth, Minnesota, and Superior, Wisconsin," by the River and Harbor Act of Congress approved on June 3, 1896.

The district engineer at that time also appeared before the river and harbor committee with reliable statistics compiled by his office in 1895. This report showed a total of 10,986 arrivals and departures carrying 6,325,351 tons valued at $95,000,000 for the combined Duluth and Superior harbors.

The story of the dredging of the artificial entrance, or the Duluth Ship Canal, originated as early as 1859. In the charter of the City of Duluth, organized in April, 1870, power was given the council to construct a canal connecting Lake Superior with Superior Bay. In the fall of 1870 the city let a contract for dredging a canal 150 ft wide and 16 ft deep, protected by piers on each side, extending into 18 ft of lake water. The work was pushed with all possible speed, and when winter put a stop to operations, dredging had been completed through about two thirds of the peninsula. On petition of interests in Superior, an injunction was issued by the United States Circuit Court restraining the dredging company and the City of Duluth from constructing the canal, but was later dissolved with the condition that the City of Duluth build a dike across the bay, making two separate harbors of Superior Bay. There followed seven years of litigation, until the United States Supreme Court decided in 1877 against the City of Superior. The dike was constructed but lasted only a short period, being destroyed by storms. The only railroad at that time ran into Duluth and the lack of a dike kept Superior commerce from easy access to the railway dock. Prior to opening the Duluth Canal, most of the Duluth docks were located on the shore of Lake Superior, and the first grain elevator at the Head of the Lakes was located on the lake front. With the opening of the Duluth Canal, development began on the Duluth side of the inner harbor.

The present Duluth Canal piers were completed in 1901. They consist of a concrete superstructure on a rock-filled timber crib substructure supported on piling, and are constructed to withstand the severest storms. The piers are 1,734 ft long and extend about 1,000 ft lakeward, beyond the shore line. The entrance channel has been improved to provide depths decreasing from 32 ft at the lake end to 26 ft at the harbor basin. The piers are 300 ft apart but have been ripråpped, thus reducing to 240 ft the width of channel having project depth. The old canal with its timber cribs was 250 ft wide and 1,200 ft long, with channel depths decreasing from 25 ft to 20 ft.

In 1901 a prominent engineer predicted that within twenty-five years the structure would have to be pulled down to provide for larger or better improvements, yet in 1947 the project was adequate for the largest vessels. Moreover, if ocean-going vessels come to the lakes as the result of an improvement of the St. Lawrence Waterway, Duluth-Superior Harbor would require only the deepening of its channels by a few feet to accommodate them.

The vertical lift bridge over the Duluth Ship Canal was authorized by an act of Congress and is owned and operated by the City of Duluth. As originally constructed in 1905, the bridge consisted of two towers and an overhead span. A car suspended from this overhead span traveled on regular trips between the approaches on each side of the canal. The capacity of the car was four vehicles per trip. In 1929 the bridge was modified by raising the fixed span 42 ft, constructing new towers within the towers of the existing bridge, and providing a lift span 393 ft 9 in. in length to handle vehicles and pedestrian traffic. The old towers and fixed span are not essential to the new bridge but are used to carry public utility pipes and cables.

The bridge has several sources of power, the main one being from storage batteries located under the south approach. These storage batteries are recharged by motor generators operated by local commercial power or by a standby, gasoline driven charging set. Double sets of trolleys are available for transferring power to the operating room of the lift span; the extra set is considered necessary to assure contact during unfavorable weather conditions, which may cause formation of ice on the trolleys. Four 100-hp motors are available in the operating cage in the lift span, any combination of two being sufficient for raising and lowering the span. A 300-hp gasoline engine, located in the operating house, in an emergency is capable of raising the bridge in 2½ min by merely shifting a gear to connect the engine to the operating mechanism. Normal time of lift is a little less than 1 min.

Vertical clearance when the span is raised is 138 ft; when down, it is 16.5 ft. The concrete counterweights each weigh 450 tons and are built with pockets to contain separate small concrete blocks to be added to adjust variation in weight of the span, particularly after a fall of heavy wet snow or after a sleet storm.

Duluth-Superior Harbor has a frontage of 49 miles behind established harbor lines; the harbor area is 19 sq miles. There are 17 miles of dredged channels with widths ranging from 200 ft to 600 ft and two anchorage areas, one in the Duluth Harbor basin, the other in the Superior Harbor basin, with ample room for anchorage of from 35 to 50 vessels.

The federal government expended a total of $10,915,400.13 in permanent work and maintenance of Duluth-Superior Harbor prior to June 30, 1946. This represents a cost of about $0.005 per ton as applied against the 2,000,000,000 tons of commerce handled by this harbor.

Coal is one item of bulk commodities wherein a large saving to the public results from water shipment over rail shipment. For example, in 1947, the rail charges on coal shipments from the Illinois mines, the nearest supply of coal to the Head of the Lakes, amounted to approximately $4.05 per ton to Duluth, as compared with the rate in 1940 of $0.40 per ton applying to the movement by water of a superior quality of coal from Lake Erie ports to Duluth-Superior Harbor. In 1940 more than 7,000,000 tons of coal were unloaded on docks at the "Twin Ports," resulting in a saving to the public of about $26,000,000 for that year. The total receipts of coal prior to July, 1947, at this harbor exceed 400,000,000 tons.

The major cargoes on the lakes are the bulk commodities of iron ore, coal, grain, and limestone, which, likewise, are the main items of commerce at Duluth-Superior Harbor. About 98% of the cargo tonnage moving through Duluth-Superior Harbor consists of bulk commodities. The majority of the vessels commonly employed on the Great Lakes are a highly specialized type of ship and are known as the bulk freighter. The loading and unloading equipment is also highly specialized to fit the ship design. The lake carriers haul more than twice the normal tonnage borne by the rest of the United States merchant fleet combined. The combined lake fleets of American and Canadian bulk cargo vessels at the close of navigation in 1946 totaled 669 steamers, motor ships, and barges of more than 2,970,000 gross tons. In 1886 about 25% of the

vessels were sailboats, and steel ships 300 ft in length were just making their appearance. The cargo vessels in the Great Lakes fleet under United States registry now consist preponderantly of vessels 600 ft or more in length—a far cry from the canoes used in the first commercial traffic recorded in the Duluth-Superior area in 1855.

Duluth-Superior Harbor consistently ranks second only to New York (N. Y.) harbor as the largest port in the United States in point of tonnage, and is probably the most important bulk traffic port in the world—despite the fact that it enjoys a navigation season of only eight months. For the calendar year 1945, the leading ports of the United States in point of tonnage were:[2]

Order	Harbor	Million short tons
1	New York, N. Y.	103.9
2	Duluth-Superior	65.4
3	Philadelphia, Pa.	37.2
4	San Francisco Bay, Calif.	32.2
5	Toledo, Ohio	30.5

In the foregoing comparison the total tonnage shown for Duluth-Superior Harbor originates at, or is destined for, localities outside the vicinity of the port, whereas all but 50,000,000 tons of the commerce credited to New York harbor is classed as local or within port commerce. The following tonnage data of commerce will give some idea of the growth of commerce at Duluth-Superior Harbor:

Year	Million tons
1871	0.093
1890	3
1910	36.5
1930	45.5
1942	74.3
1945	65.4

Shipments of iron ore constitute the largest item of commerce for the harbor; it amounted to about 39,000,000 short tons in 1946, or 71% of the total tonnage of the harbor. Of all the iron ore shipped from Great Lakes ports in 1946, 59% was shipped from the Duluth-Superior docks. During the war, freighters of the Great Lakes carried six sevenths of the ore used in making American steel. There are 308 bulk freighters engaged in the iron ore trade, almost all of which visit this harbor many times during a season. The first ore was shipped from the harbor in 1892. There are seven ore docks at Duluth-Superior with a total length of 15,216 ft and total storage capacity of 819,100 tons. The number of pockets range from 302 to 384, depending on the length of the dock. Thus, it is possible to load four or five large ore vessels at each dock simultaneously. Boats are normally loaded in three or four hours, with an average cargo of 12,000 tons of iron ore.

[2] "Annual Rept. of the Chief of Engrs., U. S. Army," U. S. Govt. Printing Office, Washington, D. C., 1945.

Receipts of coal and coke of more than 10,000,000 tons constituted 19% of the total tonnage of the harbor in 1946. A coal dealer's publication in 1935 showed that at Duluth-Superior Harbor is concentrated the world's largest reserve of prepared coals and screenings, ready for immediate shipment to points in Minnesota, Wisconsin, northern Iowa, North Dakota, South Dakota, and northeastern Nebraska. There are now 19 docks receiving coal at Duluth and Superior.

Grain, flaxseed, and screenings receipts and shipments constituted 6% of the tonnage of the harbor in 1946 and amounted to nearly 3,500,000 tons. Of this, 151,000 tons were received from Canadian lake-head ports of Port Arthur and Fort William in Ontario, and the remainder was shipped to lower lake ports. Duluth-Superior shipped 87% of the grain transported in lake movement from American ports on the Great Lakes in 1946. Lake carriers carried one third of the world's grain supply during the World War II. The sixteen elevator systems at Duluth-Superior include thirty elevators with a total storage capacity of nearly 52,000,000 bu.

In addition, vessels at this harbor are engaged in carrying limestone, petroleum products, iron and steel—including scrap iron, automobiles, cement, salt, gravel, sand, manganese ore, and various smaller items. In all, there are 110 terminals at this port.

With the present development of processes to utilize low-grade taconite ore the future of Duluth-Superior Harbor seems certain.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

SOME EFFECTS OF ANAEROBIC DIGESTION ON SEWAGE SLUDGE

BY A M RAWN,[1] M. ASCE, AND E. J. CANDEL,[2] ESQ.

SYNOPSIS

That anaerobic digestion of sewage sludge effects a marked reduction in the biochemical oxygen demand (B.O.D.) and coliform group density thereof, as well as a radical change in sludge characteristics, is well known. The extent of these phenomena, however, does not appear to have been very well explored, and factual data relating thereto have been gleaned from research leading toward other objectives.

This paper relates the results of an investigation aimed at discovering the numerical value of some of the changes occurring in digesting sewage sludge. The work was inspired by experience in the disposal of digested sewage sludge from temporary military establishments during World War II, and by public health regulations in some states to the effect that digested sewage sludge may not be discharged into state waters, fresh or salt.

Results of the investigation indicate that, where sludge banks will not be formed, there is little, if any, reason why well-digested sewage sludge may not be disposed of by dilution along with a primary, clarifier effluent if conditions will permit disposal of the latter with safety.

INTRODUCTION

The disposal of digested sewage sludge presents many perplexing problems and, at times, constitutes the major one in sewage treatment. As long as well-digested, wet sewage sludge continues to be viewed as a relatively dangerous substance, its final disposition will be presumed to be accompanied with great risk unless, prior to final disposal, it is rendered completely innocuous by the destruction of all its original, dangerous characteristics.

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by April 1, 1949.

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The purpose of this investigation is to evaluate the effect of returning digested sludge to primary effluent prior to disposal of the latter by dilution. Conclusions are drawn by measuring the changes which occur in B.O.D., bacterial population, grease, quantity of solids remaining, and other characteristics, after being digested under varying conditions, from 11 days to 80 days and then computing the effect of recombining digested sludge with primary effluent.

General Description of Experiments

Experimental work for this investigation was conducted at the joint disposal plant of the Los Angeles County Sanitation Districts, in Los Angeles, Calif. Work on the investigation was commenced on February 28, 1946, and still continues in some phases. Tests were made on sludge incubated in bottles and also on sludge taken from the digestion tanks at the disposal plant. In general, the observations of digestion phenomena were made on three series of samples.

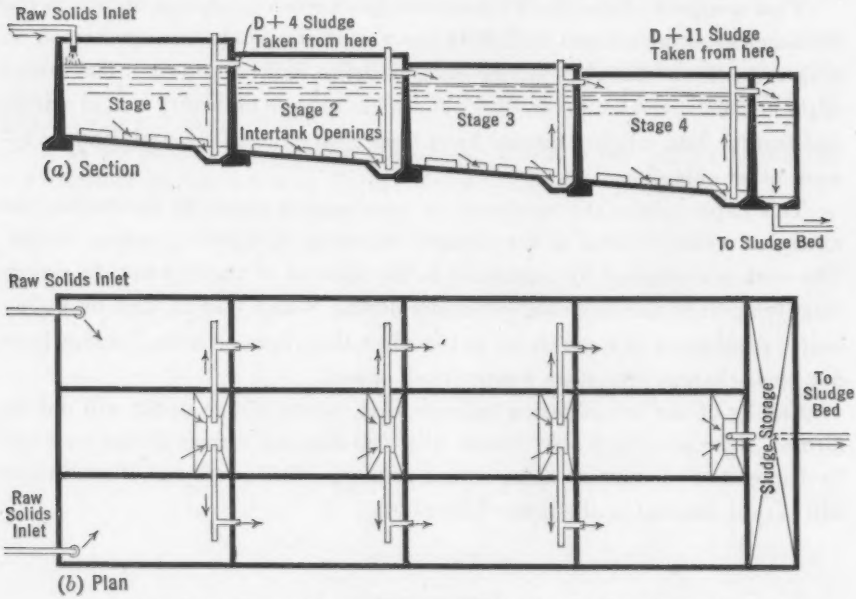

Fig. 1.—Four-Stage Sewage Sludge Digestion Tanks of the Joint Disposal Plant, County Sanitation Districts of Los Angeles County, California

Before proceeding further with the description of the experiments, attention is directed to the design of sludge digestion tanks at the districts' plant. The tanks are designed in four stages[3] with intertank piping so that sludge discharges by gravity from the bottom of the tanks in each stage into the top of the tanks in the next stage, as shown in Fig. 1. Seed sludge from the last or fourth stage is returned to the first stage. No stirring is provided other than that which results from the flow of sludge through the tanks. The formation

[3] "Multiple-Stage Sewage Sludge Digestion," by A M Rawn, A. Perry Banta, and Richard Pomeroy, *Transactions*, ASCE, Vol. 104, 1939, p. 93.

of scum mats is actually encouraged in this design. Cleaning of the tanks is necessary at from 4-year to 6-year intervals, but between cleanings the automatic segregation and retention of the lighter solids results in excellent plant performance. At the time of the tests reported herein, the average detention period of the sludge in the digestion tanks was 11 days; and that in tank No. 1, approximately 4 days.

A more detailed description of the procedure with each of the three series laboratory tests follows:

Series 1.—The ten specimens comprising this series were each composed of 83% by volume of untreated sewage sludge from primary sedimentation and 17% by volume of digesting sewage sludge from the fourth stage of the sludge digestion tanks.

Series 2.—The five specimens in this series were taken directly from the fourth stage of the digestion tanks and allowed to continue digestion for 60 days. The age of the sludge when it was taken from the tanks was assumed to be 11 days.

Series 3.—The three specimens included in this series were taken from the outlet of the first stage of the sludge digestion tanks and allowed to continue digestion until the age of 60 days. When taken from the tanks, the average age of the sludge, including recirculated seed, was estimated to be 4 days.

In each of the series, sludge digestion was instituted or continued in 2-liter bottles at 85° F under a gas pressure of about 0.33 lb per sq in., thereby simulating conditions obtainable in the districts' digestion tanks.

Five-Day (20° C) B.O.D. of Raw Sewage Solids

As determined by the method given in "Standard Methods for Examination of Water and Sewage," [4] 5-day B.O.D. results in values for sludge that appear to be neither true nor reliable. In every case the B.O.D. value obtained indicated oxygen depletion lower than that calculated on the basis of sludge removal from the sewage entering the plant. A typical test, as shown in Table 1, indicates a 5-day B.O.D. for the sample of 12,100 ppm, whereas the determination of removal of B.O.D. from the sewage passing through the treatment plant would indicate that the B.O.D. of the raw sludge should be 20,000 ppm.

TABLE 1.—Biochemical Oxygen Demand of Sludge

Sludge[a] (ml)	Depletion		B.O.D. (ppm)
	O_2 (ppm)	Available O_2 (%)	
(1)	(2)	(3)	(4)
0.06	3.70	44	15,600
0.10	4.87	58	12,100
0.12	5.19	62	10,900
0.14	5.50	66	9,780
Mean	....	..	12,100

[a] In 250-ml B.O.D. bottle.

The relative strengths of solids contained in sewage have been indicated by W. Rudolfs, M. ASCE, and H. W. Gehm,[5] and their work is accepted as a

[4] "Standard Methods for the Examination of Water and Sewage," Am. Public Health Assn., 8th Ed., New York, N. Y., 1936.

[5] "Chemical Coagulation of Sewage; IV. B.O.D. of Soluble and Dispersed Materials," by W. Rudolfs and H. W. Gehm, *Sewage Works Journal*, Vol. 8, 1936, p. 537.

general pattern to verify the conclusions that district measurements of the B.O.D. of sludge by standard methods are low, but may be corrected to give a fair and consistent value.

C. C. Ruchhoft and W. A. Moore,[6] as well as H. F. Stephenson,[7] by their experimental work indicate that without constant agitation dissolved oxygen is not continuously available to the oxygen-demanding sludge in a sludge specimen being tested for B.O.D. Because equipment for continuous agitation was not available for this experiment, it was decided to measure the B.O.D. in the usual way, with subsequent application of suitable corrections in accordance with Fig. 2.

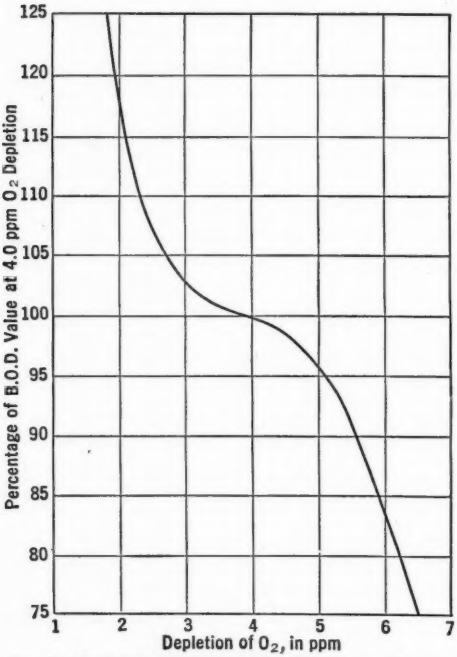

FIG. 2.—RELATIONSHIP OF B.O.D. VALUES OF SLUDGE AT VARIOUS OXYGEN DEPLETIONS TO B.O.D. VALUE AT DEPLETION OF 4.0 PPM

The most practical procedure for the purpose of this research, and one which has been used in correlating results in other district investigations, appears to be to select B.O.D. values corresponding to oxygen depletion of 4 ppm or about 50% of the available oxygen in the incubated mixture. The results so obtained are probably lower than would be found if the most favorable conditions for biochemical oxidation could be maintained; on raw sludge the results appear to be about 75% as great as might be expected from the B.O.D. indicated by analyses of plant influent and effluent. This, however, is not overly important for the purposes to which the tests are put. All conclusions herein are based on percentage reductions of B.O.D. of the sludge, and these percentages are valid as the various B.O.D. results are comparable with one another. It is believed that the arbitrary selection of a depletion of 4 ppm as standard gives results which are comparable.

CHANGES IN B.O.D. DURING DIGESTION

The changes occurring during digestion in 5-day B.O.D. of the three series of sludge samples are shown in Fig. 3. In series 1 (Table 2) the B.O.D. value of the sludge, after dropping to a temporary low on the fourth day, rose by the twentieth day to a value higher than that initially indicated and then declined

[6] "Determination of Biochemical Oxygen Demand and Dissolved Oxygen of River Mud Suspensions," by C. C. Ruchhoft and W. Allan Moore, *Industrial and Engineering Chemistry*, Analytical Edition, Vol. 12, 1940, p. 711.

[7] "Water Analysis; Effect of Suspended Matter and Daylight on Incubation Tests," by H. F. Stephenson, *Analyst*, Vol. 64, 1939, pp. 344–346.

steadily. In series 2 (Table 3) substantially all that had been accomplished in digestion in series 1 in 80 days was accomplished in the stage digestion tanks in 11 days, and the same was substantially true for series 3 (Table 4), for which

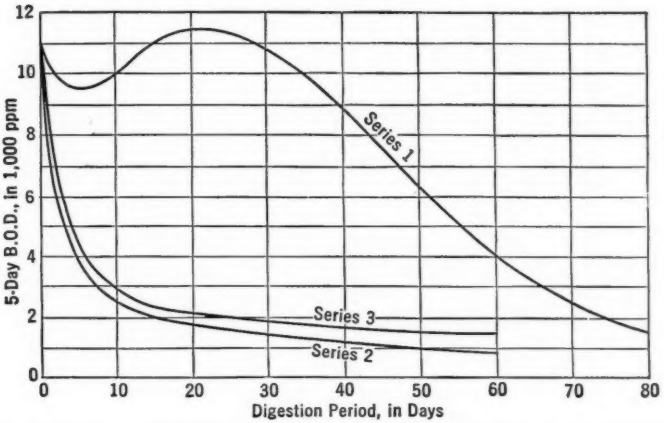

FIG. 3.—CHANGES OCCURRING IN 5-DAY B.O.D. DURING DIGESTION OF SLUDGE SAMPLES

TABLE 2.—OBSERVED 5-DAY BIOCHEMICAL OXYGEN DEMAND FOR SERIES 1

Specimen No.	5-DAY B.O.D. OF SLUDGE AT DIGESTION PERIOD, IN DAYS								
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	10,800[a]	7,000	10,100	8,900	2,900	1,400	1,400	1,200	1,200
	0	10	20	30	40	50	60	72	80
2	10,700	10,100	11,900	13,600	10,400	10,000	3,900	2,100	1,400
	0	10	20	30	40	50	63	70	83
3	12,800	9,400	13,100	10,400	11,800	11,000	8,300	3,800	1,900
	0	10	20	30	40	50	60	72	81
4	13,100	12,200	11,300	11,000	11,600	11,000	6,000	2,300	1,500
	0	10	20	30	40	50	60	70	79
5	9,300	9,000	10,000	9,700	10,100	11,400	9,300	3,200	....
	0	10	20	30	40	50	60	82	....
6	13,900	11,500	15,300	10,300	9,600	2,100	1,400	1,100	....
	0	10	20	30	40	60	73	80	....
7	10,200	10,200	10,600	10,500	9,300	5,000	2,000	1,500	1,200
	0	2	12	22	32	42	51	64	82
8	9,700	7,700	10,600	9,000	6,600	1,500	1,400	1,000	....
	0	8	18	28	38	62	70	80	....
9	10,600	7,400	12,000	11,600	10,200	11,000	9,700	6,100	2,900
	0	6	16	26	36	47	60	68	78
10	9,900	6,400	11,000	9,100	8,800	4,000	1,000	850	....
	0	4	14	24	34	43	56	74	....
Mean	11,100	....	....	....	....	....	....	....	1,600±
	0	....	....	....	....	....	....	....	80

[a] Upper value is 5-day (20° C) B.O.D. of digesting sludge; lower value, digestion period in days.

the samples were taken after 4 days in the first stage of the four-stage digestion tanks.

This series of experiments was not performed to indicate the efficiency of four-stage digestion or any other type of digestion but to illustrate the very

wide difference between series 1, on the one hand, and series 2 and 3 on the other. It should impress the investigator with the time required for adequate digestion in functioning digestion tanks, as compared with the time which might be required in starting from scratch with bottle specimens.

TABLE 3.—OBSERVED 5-DAY BIOCHEMICAL OXYGEN DEMAND FOR SERIES 2

Specimen No.	Age of Sludge, in Days						
	0	11	17	24	30	45	60
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	....	2,040	1,570	1,200	1,160	1,010	980
2	....	2,000	1,460	1,470	1,330	1,160	1,000
3	....	2,660	1,700	1,300	1,330	950	990
4	....	2,640	1,550	1,910	1,000	860	850
Mean	11,100[a]	2,340	1,570	1,220	1,200	995	955

[a] Average B.O.D. of sludge entering digesters.

In series 1 it is clearly indicated that after an initial drop in B.O.D. there is a subsequent rise to a value greater than that initially determined. Although this series of experiments was not conducted to determine the reasons for this occurrence, speculation indicates that, after a short period of digestion, some of

TABLE 4.—OBSERVED 5-DAY BIOCHEMICAL OXYGEN DEMAND FOR SERIES 3

Specimen No.	Age of Sludge, in Days					
	0	4	7	11	30	60
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	....	6,300	4,000	3,000	2,200	1,800
2	....	4,700	3,600	2,600	1,400	1,800
3	....	4,600	3,000	2,300	1,200	900
Mean	11,100[a]	5,200	3,530	2,630	1,600	1,500

[a] Average B.O.D. of sludge entering digesters.

the compounds initially resistant to bacterial oxidation may have changed to the extent of rendering them more readily available as bacterial food—for example, by the well-known conversion of certain insoluble material to soluble volatile organic acids. This supposition is verified partly by the continuous and unchecked reduction in organic solids throughout the experiments in series 1 despite the fluctuating B.O.D. values. It is assumed that the conditions in the stage digestion tanks and in other functioning digestion tanks either forestall the rise of B.O.D. noted in series 1 or else mask it. The work of R. S. Ingols and G. M. Ridenour,[8] Assoc. M. ASCE, indicates that B.O.D. changes in raw sewage stored anaerobically varied in an unpredictable manner.

[8] "Effect of Storage of Sewage Upon Its B.O.D.," by R. S. Ingols and G. M. Ridenour, *Sewage Works Engineering and Municipal Sanitation*, Vol. 17, 1946, p. 315.

Examination of the data for series 3 discloses a marked reduction in B.O.D. values in 4 days, and a fairly consistent reduction until the age of 60 days is reached. Comparison of the three series indicates the best reduction for series 2, something less for series 3, and the least effective for series 1.

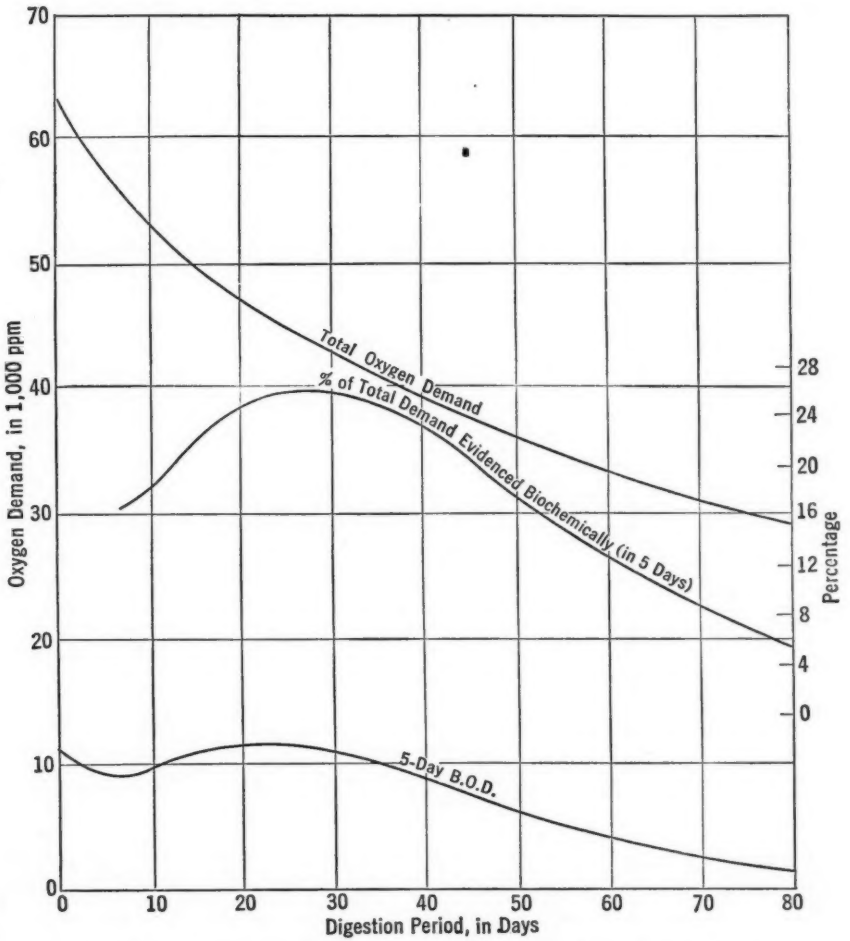

FIG. 4.—RELATIONSHIP OF B.O.D. TO TOTAL OXYGEN DEMAND OF SERIES 1 SLUDGES DURING DIGESTION

As a check on the observed B.O.D. values for series 1, the total oxygen demand of the digesting mass is calculated, using as a basis an estimate, taken from the unpublished notes of Richard Pomeroy, which indicates that each part of the organic matter in the digesting sludge requires 2.1 parts of oxygen for its complete oxidation. Fig. 4 illustrates the conclusion that at all times the apparent 5-day B.O.D. in series 1 was much less than the total oxygen demand of the digesting mass.

The percentage of total oxygen demand exerted biologically in 5 days reached a maximum of 26% between 25 days and 30 days. If 33% is added to the B.O.D. values to allow for error in the method of determination, and, if it is assumed further that this 5-day value is 68% of the "ultimate first-stage B.O.D.," it appears that a little more than one half of the organic matter at this stage is readily susceptible to biological oxidation. At the end of 80 days only 11% of the remaining organic matter is similarly available for bacterial food.

The B.O.D. of the sludge in series 2 was reduced from 11,100 ppm (observed B.O.D.) to 955 ppm, or 91.5% in 60 days. At the Los Angeles districts' disposal plant, sludge is removed from raw sewage in the ratio of 1 part wet sludge (at about 96% moisture) from 208 parts sewage. Estimating a sludge volume reduction (by removal of supernatant liquid and by reduction of volatile matter) of 40% at the end of 60-day digestion, the volume of digested sludge bears the ratio to the primary effluent of 1 part digested sludge to 350 parts effluent.

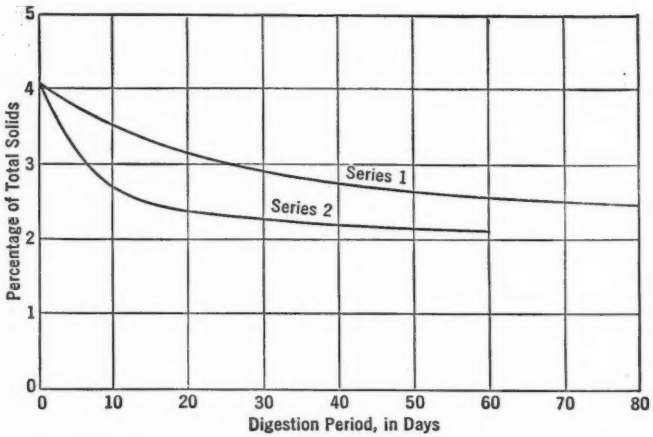

FIG. 5.—COMPARISON OF CHANGES IN TOTAL SOLIDS IN TWO SERIES OF SLUDGE SAMPLES DURING DIGESTION

On the basis of corrected values, the B.O.D. of the digested sludge appears to be 1,275 ppm (measured 955 ppm). The average B.O.D. of the effluent during the investigation was 150 ppm. Therefore, the return of the digested sludge to the primary effluent would increase the B.O.D. of the latter by 2.1%, or from 150 ppm to 153.2 ppm.

SOLIDS REDUCTION DURING DIGESTION

Total solids were determined by weighing and drying a 20-g portion of sludge in the conventional manner. Solids so determined are probably lower than those that are actually present due to the loss of volatile organic matter and ammonium carbonate during the drying process. However, the results are used as so determined. Ash was identified by ignition of the dry solids

in an electric furnace at dull red heat for 1 hour. Analytical results for total solids and ash are shown in Tables 5 and 6, and are presented graphically in Fig. 5.

The observations with respect to B.O.D. behavior during the test run are not paralleled in the observations of solids reduction. Series 2 fared better

TABLE 5.—SOLIDS DATA FOR SERIES 1

Specimen No.	Ash (%)	TOTAL SOLIDS IN SLUDGE AT DIGESTION PERIOD, IN DAYS (%)								
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	0.94	3.60[a] 0	3.02 10	3.02 20	2.84 30	2.47 40	2.25 50	2.19 60	2.16 72	2.16 80
2	1.07	4.26 0	3.47 10	3.49 20	3.33 30	3.26 40	3.10 50	2.86 63	2.62 70	2.62 83
3	1.51	4.85 0	4.51 10	4.09 20	3.98 30	3.76 40	3.60 50	3.42 60	3.14 72	3.19 81
4	1.22	4.80 0	4.18 10	3.96 20	3.63 40	3.48 50	3.17 60	3.02 70	2.90 79	....
5	1.03	4.40 0	3.90 10	3.66 20	3.34 30	3.32 40	3.22 50	3.12 60	3.18 70	2.74 82
6	1.22	4.73 0	4.69 10	3.88 20	3.78 30	3.44 40	3.10 53	2.97 60	2.92 73	2.88 80
7	1.06	3.96 0	3.72 2	3.34 12	3.26 22	3.07 32	2.78 42	2.56 64	2.34 72	2.44 82
8	0.83	3.46 0	3.09 8	2.84 18	2.73 28	2.72 38	2.57 49	2.26 62	2.20 70	2.20 80
9	0.95	3.44 0	3.20 6	3.10 16	3.04 26	3.12 36	3.08 47	2.86 60	2.54 68	2.36 78
10	0.88	3.48 0	3.22 4	2.92 14	2.80 24	2.78 34	2.46 43	2.28 56	2.10 64	....
Mean	1.07	4.10 0	....	....	....	....	....	....	....	2.46± 80

[a] Upper value is percentage of total solids; lower value, digestion period in days.

TABLE 6.—PERCENTAGE OF TOTAL SOLIDS FOR SERIES 2

Specimen No.	Ash (%)	AGE OF SLUDGE, IN DAYS						
		0	11	17	24	30	45	60
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	0.94	....	2.56	2.32	2.28	2.24	2.20	2.14
2	0.93	....	2.52	2.32	2.30	2.26	2.07	1.97
3	1.13	....	2.78	2.66	2.56	2.50	2.48	2.32
4	0.97	....	2.76	2.46	2.34	2.33	2.26	2.00
Mean	0.99	4.19[a]	2.66	2.44	2.37	2.33	2.25	2.11

[a] Average solids content of raw sludge for period, corrected for 11% over-all seeding with digested sludge.

than series 1, and series 3 was the least affected. Comparison of percentage reduction in volatile matter in series 1 and 2 is shown in Fig. 6. Results of series 3 are not shown because of their erratic nature. In part they paralleled series 2 and in part they varied widely. It is thought that the methods of taking samples from intertank connections and of calculating the original

solids content are responsible for the nature of the results obtained with series 3.

The total solids content in series 2 was reduced from 4.19 to 2.11, or 49.6%. Assuming that all this would be settleable when returned to the primary effluent, its return in the ratio of 1:350 would raise the average settleable solids of the primary effluent by 1.5 ml per liter.

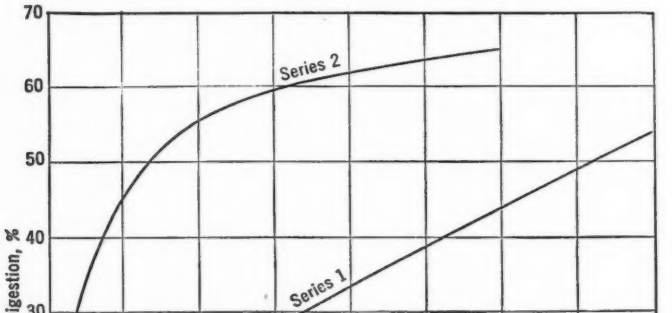

FIG. 6.—COMPARISON OF PERCENTAGE REDUCTION IN VOLATILE MATTER IN TWO SERIES OF SLUDGE SAMPLES DURING DIGESTION

TABLE 7.—COLIFORM BACTERIA PER MILLILITER

Specimen No.	AGE OF SAMPLE, IN DAYS			
	0	11	30	60
(1)	(2)	(3)	(4)	(5)
1	....	7,000	2,400	1,300
2	....	6,200	2,400	600
3	....	7,000	6,200	2,400
4	....	7,000	600	600
5	....	7,000	2,400	600
Mean	1,504,000[a]	6,800	2,800	1,100

[a] Average coliform group density of raw sludge for test period, corrected for 11% over-all seeding with digested sludge.

REDUCTION IN COLIFORM GROUP DENSITY

The most probable number (M.P.N.) of coliform group bacteria in each test was determined by making lactose broth presumptive and brilliant green bile broth confirmatory tests in accordance with aforementioned procedures.[4] Each M.P.N. was based on paired innoculation dilutions from 10^{-2} to 10^{-7} as indicated in determinations on five samples shown in Table 7.

During the period of this study, primary effluent averaged 51,000 coliform group bacteria per ml. At the age of 60 days the coliform count of digested

sludge from series 2 averaged 1,100 per ml, whereas at 11 days the average was 6,800 per ml. It is evident then that the return of 11-day sludge or 60-day sludge from the stage digesters would actually result in a dilution of coliform bacteria and that primary effluent will probably suffer no degradation in this characteristic by the addition of digested sludge.

Reduction in Grease Content During Digestion

Raw solids from the untreated sewage averaged 16,000 ppm (wet basis) grease during the investigation. Grease determinations on raw and digested sludge were obtained by acidification with sulfuric acid, drying, and extraction with ethyl ether.

TABLE 8.—Grease Reduction Data, as Total Grease in Parts per Million (Wet Basis)

Specimen No.	Age of Sample, in Days		
	11	30	60
(1)	(2)	(3)	(4)
1	....	....	2,210
2	....	....	2,480
3	....	....	2,840
4	3,930	2,770	2,650
5	3,450	2,580	2,410
Mean	3,690	2,680	2,500

Grease determinations on sludge samples taken from the outlet of the fourth stage of the districts' sludge digestion tanks are presented in Table 8, Col. 2. The grease determinations were inaugurated after the experimental work was well along and, consequently, are not all made on original samples reported in series 1, 2, and 3. However, the tests made on 30-day sludge and 60-day sludge, shown in Table 8, are felt to be sufficiently typical to indicate the numerical value of grease reduction.

Primary clarifier effluent averaged 34 ppm of grease by acidification and direct hexane extraction during the period of this investigation. Table 8 indicates that sludge from stage digestion at 60 days has a grease content of about 2,500 ppm. If the digested sludge at 60 days is returned to the primary effluent in the ratio of 1:350, the grease content of the latter will be increased from 34 ppm to 41 ppm, or approximately 21%. Similarly, if sludge from the stage digestion tanks with an age of 11 days is returned to primary clarifier effluent, the grease content will be increased from 34 ppm to about 44.5 ppm.

Effect of Digestion on Chlorine Demand

For the purpose of this investigation, the chlorine demand is defined as the parts per million of chlorine added to diluted sludge which are required to produce an apparent residual chlorine content of 0.1 ppm after a 15-min contact period. Dilution of the sludge was required in order to observe color changes. Chlorine demand of raw sludge during the investigations varied from 950 ppm to 1,000 ppm. The 60-day-old sludge had a demand of about 600 ppm, whereas primary effluent had a demand of 36 ppm. If the 60-day-old digested sludge from series 2 were returned to the effluent in the ratio of 1 part of the former to 350 parts of the latter, the chlorine demand of the effluent would be raised to 37.7 ppm, or about 4.7%.

Discussion

Each case involving the disposal of digested sludge with effluent from a primary subsidence treatment process must be considered on its own merits and conclusion 5, stated hereinafter, should be interpreted with special emphasis on the statement, "wherever sludge banks will not be formed." This qualification removes the conclusion from a generalization, but indicates that, under certain conditions, there are situations in which the disposal of digested sludge with primary effluent should be seriously considered as having no public health or nuisance implications and as effecting major economies in sewage treatment and disposal.

One of the major reasons for primary treatment of sewage is to prevent the formation of deposits of raw sewage solids commonly termed "sludge banks." Such deposits will putrefy and cause foul odors if they become exposed to the air; they may release hydrogen sulfide to overlying bodies of water; and they may cause an oxygen demand on the receiving stream greater than is indicated by the B.O.D. of the raw sewage, because the solids may be retained in the vicinity of the point of discharge until a relatively large fraction of the ultimate B.O.D. is satisfied.

These objections largely disappear as the sludge is taken out, digested, and then returned to the effluent. However, the authors would be loathe to conclude that disposal of the digested sludge with primary effluent to a stream, to a lake, or to tidewater is justified where sludge banks of even well-digested sludge will accumulate—hence, the qualification in conclusion 5.

Bacterial pollution of the receiving waters by raw sludge is also a matter of concern. Chlorination of raw sewage is not likely to be 100% effective because the bacteria are sheltered by solids and sludge particles. If, however, the sewage is settled, the effluent may be effectively chlorinated, and if the sludge is digested the destruction of intestinal bacteria is so great that with digestion periods of from 30 days to 60 days under most conditions (and shorter periods in controlled tanks), it appears that the sludge can be returned to the effluent with no more health hazard than is inherent in the disposal of primary effluent, even when the latter is chlorinated.

In the disposal of sewage into the ocean an important reason for primary treatment is to eliminate floating material. Skimming alone will not accomplish this satisfactorily, because some solids carried down by the sludge into the sludge banks may later break loose and come to the surface. In the digestion process the sludge is disintegrated so that substantially all the floatable matter can be separated. If such sludge (minus scum-forming solids retained in the digestion tanks) is discharged with the effluent, no nuisance from floating solids should result.

Conclusions

1. Under digestion conditions such as those existing at the Los Angeles County Sanitation Districts' joint disposal plant, addition of sewage solids that have been digested for 60 days (the first 11 days of which have been spent

in the four-stage digestion tanks) to primary clarifier effluent (the latter by itself having an average 5-day (20° C) B.O.D. of 150 ppm) will, on the average, change the characteristics of the primary clarifier effluent in a manner similar to the following:

(*a*) Increase the 5-day (20° C) B.O.D. by 3.2 ppm, or 2.1%;
(*b*) Increase the settleable solids by 1.5 ml per liter;
(*c*) Slightly decrease the coliform bacterial population;
(*d*) Increase the grease content from 34 ppm to 41 ppm; and
(*e*) Increase the chlorine demand from 36 ppm to 37.4 ppm, or 4.7%.

2. Substitution of 11-day-old sludge from the districts' four-stage digestion process for 60-day-old sludge will not so materially alter the foregoing conditions as to have any appreciable effect on the characteristics of the mixed effluent and digested sludge.

3. The increase in grease content is probably not of such an order as to influence the appearance of slick at an outfall site.

4. In general, the influence of adding 11-day-old or 60-day-old sludge to primary clarifier effluent is probably less than daily variations in the character of the effluent itself.

5. Wherever sludge banks will not be formed, there appears to be no valid reason why 11-day-old, or 60-day-old, digested sewage sludge may not be disposed of by dilution along with primary clarifier effluent if conditions permit disposal of the latter with safety.

Acknowledgments

Acknowledgment is made to Richard Pomeroy, consulting chemist, and to H. W. Jorgensen, Assoc. M. ASCE, designing engineer, both of the Los Angeles County Sanitation Districts, for their help in this research work and in the preparation of this paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

MINERAL LAND SURVEYS

By Arthur J. McNair,[1] Assoc. M. ASCE

Synopsis

For the first time in 40 years, there is presented herein a discussion of the history, methods, and instructions for making mineral land surveys in the United States. Public land surveys, although relatively simpler, have been rather thoroughly treated in modern technical literature. Nevertheless, few people realize the importance of mineral land surveys from the standpoint of the number of such tracts of ground or from the standpoint of the entirely different and rather complicated procedure of surveying or resurveying mineral lands. Not only the methods of making a mineral location are presented, but also the importance of having a competent surveyor or engineer make such a survey is emphasized. The changes in regulations and their effect on the survey, together with the different methods of surveying, are rather completely discussed to assist an ordinary land surveyor in resurveys of mineral land wherever tracts of this type occur in public or private ownerships.

Introduction

The survey of mineral bearing lands in the United States for the purpose of segregating them from the public domain so that an individual may obtain a title or "patent" differs from the survey of other public lands in several respects. First, public land surveys are made by the federal government through the General Land Office, United States Department of the Interior, following a regular procedure, whereas mineral land may be originally described by anyone and the official patent survey is made by mineral land surveyors employed by the claimant and working under conditions which vary from place to place. Second, because of the greater value, or, at least, the greater potential value of the land, the precision required for a mineral land survey is higher than that required for ordinary public land surveys. As most mineral land is located in mountainous areas, the difficulty of maintaining the requisite precision is greatly increased. Third, the problem of obtaining, placing, and maintaining

Note.—Written comments are invited for immediate publication; to insure publication the last discussion must be submitted by April 1, 1949.

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adequate survey monuments is frequently difficult, and the restoration of "lost" corners requires considerable patience and judgment. Finally, mineral land surveys are complicated by the conflicts between claims, by the ties to and discrepancies in the public land surveys, and by legal procedures and court decisions, which must be properly interpreted and followed.

This paper deals with the methods employed in making location and patent surveys to segregate mineral lands from the public domain so that they may be patented and also with the important historical developments that have affected surveying procedures. During the days of the westward expansion in the United States, most of the area in the midwest and in the great plains was absorbed by the familiar procedure of homesteading, so that the lands were privately owned before minerals were discovered. With the discovery of gold in California and the subsequent mineral discoveries in other Pacific and Rocky Mountain states, the type of land development and the purposes and problems of acquisition changed. All minerals, including even building stone, were included under the original "mining acts" of Congress. The states of Kansas, Missouri, Wisconsin, Michigan, and Minnesota, as well as Texas, were excepted from these general mining acts. Various acts of Congress have restricted the lands which may be patented. For instance, the Leasing Act of 1920 permits only the leasing of land from the United States government for the purposes of prospecting for and extracting coal, phosphates, oil and gas, oil shale, sodium, sulfur, and potash. Also, since 1935 patents for homesteads and for stock raising homesteads have been essentially stopped, so that lands in the public domain which contain metalliferous minerals are about the only lands which a person can freely prospect and develop by individual enterprise with assurance of being able to obtain title to the land so developed. There are in Alaska and the states of Arizona, California, Colorado, Idaho, Montana, Nevada, New Mexico, Oregon, South Dakota, Utah, Washington, and Wyoming more than 90,000 official mineral land surveys, comprising nearly a quarter of a million separately surveyed, monumented, and patented tracts of land, many of which, because of conflicts, may consist of from one to a dozen or more small fractions of land in the original patent. Of course, as is the case with any land, many of these original tracts have been further subdivided. Inasmuch as any civil engineer, land surveyor, mining engineer, or land appraiser is likely to have occasion to resurvey or to subdivide these tracts in these states, it is important for him to have some grasp of the principles of mineral land surveys as presented herein.

Types of Mineral Claims

There are three recognized types of mineral land claims, classified according to the method of occurrence of the mineral deposits or according to the proposed use of the land: Lode claim, placer claim, and mill site.

Lode Claim.—Of these three the most important is the lode claim, which is laid out to cover mineral deposits that are "in place" in solid rock formations. A lode claim may be a maximum of 1,500 ft long and 300 ft on either side of the center line, a total of 600 ft wide, or 20.661 acres, as established by United

States statute in 1872, although the size may be limited to something less than this maximum by state or local rulings. It is preferable, where there are no conflicts with other claims, to survey the lode claim so that it will be rectangular (Fig. 1(*a*)). On many occasions, however, prior claims may have left irregular fractions of land. These can best be covered by a 600-ft by 1,500-ft quadrilateral which is not a rectangle (Fig. 1(*b*)) or by a six-sided or an eight-sided claim (Fig. 1(*c*)). Similarly, when the mineral deposit does not follow a straight line on the surface, to keep all the vein within the limits, the claim must be bent

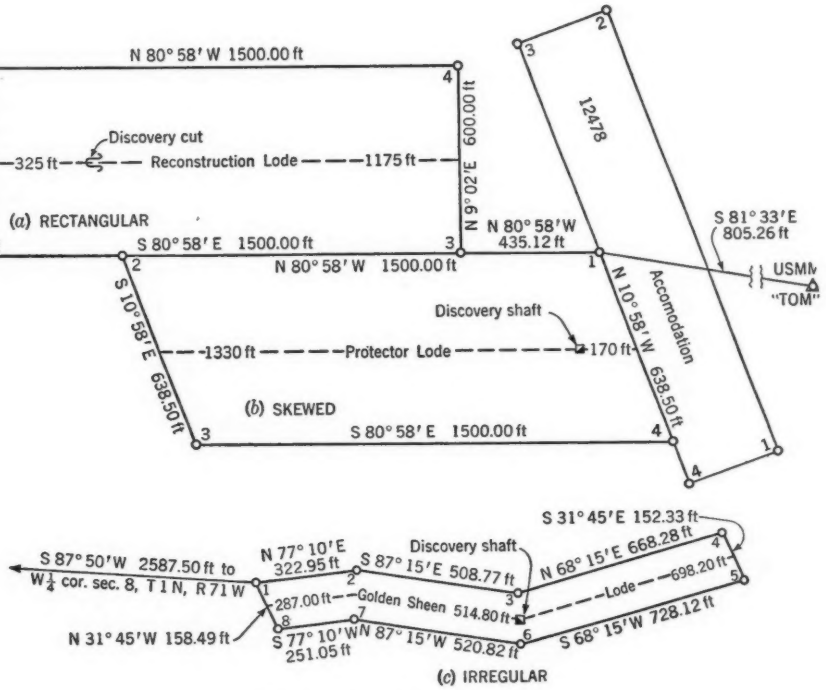
N 80° 58′ W 1500.00 ft
S 9° 02′ E 600.00 ft
N 9° 02′ E 600.00 ft
Discovery cut
325 ft
Reconstruction Lode
1175 ft
(a) RECTANGULAR
S 80° 58′ E 1500.00 ft
N 80° 58′ W 435.12 ft
12478
S 81° 33′ E 805.26 ft
USMM
"TOM"
N 80° 58′ W 1500.00 ft
S 10° 58′ E 638.50 ft
Discovery shaft
1330 ft
Protector Lode
170 ft
N 10° 58′ W 638.50 ft
Accomodation
(b) SKEWED
S 80° 58′ E 1500.00 ft
S 31° 45′ E 152.33 ft
S 87° 50′ W 2587.50 ft to W¼ cor. sec. 8, T 1 N, R 71 W
N 77° 10′ E 322.95 ft
Discovery shaft
S 87° 15′ E 508.77 ft
N 68° 15′ E 668.28 ft
287.00 ft
Golden Sheen
514.80 ft
Lode
698.20 ft
N 31° 45′ W 158.49 ft
S 77° 10′ W 251.05 ft
N 87° 15′ W 520.82 ft
S 68° 15′ W 728.12 ft
(c) IRREGULAR

FIG. 1.—TYPES OF LODE CLAIMS

so that tracts of six, eight, or even more sides result (Fig. 1(*c*)). The maximum width or the perpendicular distance between sides is the 600-ft dimension, and the length along the lode line is 1,500 ft; hence the actual lengths of sides or ends may differ from these values, as shown in Figs. 1(*b*) and 1(*c*). It is imperative that the surveyor lay out the lode claim so that the end lines are parallel.

Placer Claim.—The next most important type of mineral land claim is the placer claim, which covers mineral deposits that are not in place—for example, deposits which may be mined by washing, screening, or otherwise separating loose unconsolidated soils, sands, or gravels. Entries for placer claims, as for lode claims, are made on the basis of a discovery of mineral. However, there are two important differences in the filing of the claim—first, the discovery

must be not just a discovery of mineral, as with the lode claim, but it must be a discovery of commercial deposits of placer material such as gold, silver, platinum, sand, or gravel; and, second, the maximum amount of land which may be taken is 20 acres for one individual, increasing by 20 acres for each copartner to a maximum of 160 acres where eight or more individuals are in partnership on the claim.

There are two types of placer claims, each of which is restricted by the foregoing principles. The first type is taken up by legal subdivision of surveyed public lands. To locate this type, the surveyor must be familiar with the rules for the survey and the subdivision of United States public lands. Most civil engineers and surveyors are familiar with the procedures of rerunning and subdividing a section, and there are several good reference books on the subject.[2,3] Consequently, no further description will be given. Legally, tracts as small as 10 acres may be described by the rectangular system. To take up the desired area, a series of adjacent 10-acre tracts is described. The second type of placer claim is called the gulch placer. This type is used preferably in unsurveyed parts of the public domain where the land has not been sectionized, but is used particularly where the valuable placer deposits are confined to a comparatively narrow strip of land, such as the bed of a well-defined gulch. Claims of this type may be quite long and narrow, more than fifty sides and an over-all length of 2 miles or 3 miles being not uncommon for a 160-acre gulch placer. Maintaining the required precision on these long surveys through the timber and the undergrowth and along the steep slopes which many gulches possess is a problem which taxes the skill of the engineer.

Mill Site.—The last type of mineral land claim is the mill site, which is limited to a maximum area of 5 acres of supposedly nonmineral bearing land and may be taken up in conjunction with a lode claim or as the site of a mill or reduction works. The mill site, which should be reasonably regular in shape, is used as an adjunct to the mining, storing, milling, or treatment of ores from a lode claim.

It should be understood that one claimant, be he an individual, a group of individuals, or an incorporated body, may locate as many claims as he may deem desirable as long as he fulfils all the requirements as though each claim were an entity in itself.

Location Surveys for Lode Claims

In the early days of mining in the west, the prospector who made a discovery of mineral would post a notice by his discovery hole and proceed to lay out his claim 300 ft by 1,500 ft in size, or whatever the limit was in his particular district. He might look at the sun for directions and pace the distances, putting in his corner stakes about where he thought they belonged. Some prospectors used a compass for obtaining bearing; they might or might not allow for magnetic declination. To locate the position of the claim, bearings were usually taken on one or more mountain peaks or on other prominent natural objects

[2] "Manual of Instructions for the Survey of the Public Lands of the United States," Bureau of Land Management, U. S. Dept. of the Interior, U. S. Govt. Printing Office, Washington, D. C., 1947.

[3] "Public Land Surveys, History, Instructions, Methods," by Lowell O. Stewart, Collegiate Press, Inc., Ames, Iowa, 1935.

from one or more corners of the claim. Occasionally the lands would be surveyed lands where the section corners could be found; if so, the prospector would tie to a section corner. Probably not 10% of these locations made without the services of a qualified surveyor are truly defensible in case of legal action, and since 1920 the majority of location surveys have been made and the descriptions have been written by engineers.

Legal Requirements for Lode Locations.—The surveyor who undertakes to make a location survey must have in mind the laws and their interpretation by courts so that he will make a legal location. Prior to 1866, the length and the width of lode claims were regulated entirely by local rules. In 1866, the first act of Congress to recognize mining claims as such permitted a maximum length of 3,000 ft. The width was not specified except by local legislation. The Act of Congress of May 10, 1872, the regulation which still prevails in 1948, provided that lode claims could be a maximum of 1,500 ft long and not more than 300 ft nor less than 25 ft on each side of the lode line. These maximum dimensions were frequently restricted by state or local statutes, although since 1930 there has been no place in the United States or Alaska where the dimensions of a lode claim are restricted to less than the federal limits of 300 ft on each side of the vein, lode, ledge, or lead unless limited by conflicts with prior claims.

Discovery Hole.—The surveyor must know that a legal discovery hole should have "such a showing of mineral as would warrant a prudent miner in prosecuting the work with the full expectation of finding pay ore." This discovery must be in solid formation and not in alluvium deposits. It consists of an exposure of a vein, lode, ledge, or lead of mineral to a depth of at least 10 ft below the surface—(hence the term "10-ft hole") and may be a shaft, tunnel, open cut, trench, or even a stripping underground in some mine workings. In this last case, a survey must be run through the mine workings and out onto the surface to a point directly above the point of discovery underground, where directions for reaching the true point of discovery should be posted. At the discovery point, the judgment of the surveyor, and particularly his experience with the minerals and their occurrence in that particular district, may enable him to counsel his client wisely on the advisability of continuing the work.

Apex Rights.—A mineral deposit may occur either as a fissure vein which has an apex or as an outcrop near or on the surface, and extends an indefinite depth into the crust of the earth, usually in igneous rock formations. A mineral deposit may also occur as a "blanket" deposit along seams in, or as replacements of, sedimentary rocks, and may be thought of as replacements or fills of ancient caverns as distinguished from the narrow crack-like fissure veins.

The mineral surveyor, through his familiarity with ore deposits in the area, should recognize the type of the particular deposit he is locating for his client. If he is locating a claim on a "blanket" deposit, vertical planes passing through the side lines and end lines of the claim define the claim. However, if the claim is being located on a fissure vein, the 1,500-ft length should run in the direction of the vein because this entitles the owner to extralateral rights. These extralateral rights allow the owner of the apex of a fissure vein to follow down the dip of the vein as far as he may desire as long as he remains within the end

lines, or the end lines extended, of his claim, no matter how far this procedure may carry him beyond the vertical projection of his side lines and under other property. If the end lines of the claim are not essentially parallel, the apex owner loses his rights to mine beyond the vertical projection of his side line and may expend considerable money in developing a mine for his neighbors Needless to state, considerable litigation develops when slight variations from the standard case arise, and most of this litigation is avoidable if the surveyor knows the interpretations of the courts. An excellent treatment of various apex situations, together with a summary of various court decisions on the subject is given elsewhere.[4]

Laying Out the Claim.—If a well-defined vein can be followed on the surface for 1,500 ft, the problem of laying out the claim is simple. The surveyor selects the lengths he desires either way from the discovery point, with a maximum total of 1,500 ft, and runs out the end lines not more than 300 ft on either side normal to the center. In the event that the vein bends so that it would not be contained within the claim, then angle points are put in the side lines, and a claim with six or more sides results (Fig. 1(*c*)). To protect the apex rights of the claimant, it is essential that the end lines of the claim be parallel. If the vein and the occurrence of ore deposits cannot be definitely ascertained at the discovery hole, the surveyor lays out the claim to cover the most desirable land as the owner may decide.

Conflicts with Other Claims.—The lode location is rather simply made if there are no adjacent conflicting claims. However, this lack of conflict is frequently not the case. The surveyor must be aware of the position of any claims in the immediate vicinity which the government has previously patented to prior claimants. It is readily understood that whatever area is patented by the government to an individual or individuals becomes inviolately his or theirs. This fact, however, does not preclude a second claimant from locating a claim which may extend into or even completely across this prior claim as long as the discovery point for the new claim is on open ground. A patent can be obtained to the land within the boundaries of the new claim which is not already patented. A third claim may conflict with both of these prior claims and so on, until, what with the many directions and amounts of overlap of prior claims, the remaining fractions of unpatented land are small and widely scattered. Claims have been known to have conflicts with more than fifty other prior lodes.

Occasionally a patented lode will exclude some part of the area within its limits. In this case, a study of a district map showing the officially surveyed claims will not reveal that the area apparently covered is really not patented at all and hence is open to location. This occasionally occurs when the prior claim is abandoned for some reason or other and reverts to public domain.

Amended Locations.—Occasionally the surveyor may have to amend a location originally made when there were restrictions on the dimensions. This amendment may be necessary to reduce the total number of claims required to cover the ground or to change the location for the purpose of taking in more

[4] "Mining Rights on the Public Domain," by E. D. DeSoto and A. P. Morrison, Bender-Moss Co., San Francisco, Calif., 16th Ed., 1936.

ground up to the present legal limit. The choice as to which claims would be abandoned would depend on the showing of mineral in the various discovery holes. In the past, not infrequently, locations lay in two districts whose local regulations differed. In this case, each part of the claim was required to conform to the local regulations for the district in which it lay. In case the claims are not patented, the surveyor may have to extend the width of the claims prior to the patenting process. For the case of isolated claims mostly surrounded by previously patented claims, the problem of how to amend the location to take in the most useful ground under the new limiting conditions must be solved for each individual case. A location may be rotated about the discovery point to cover more open ground, if this seems desirable.

Equipment and Methods.—Considering the foregoing principles as to location, size, position, and conflicts of lode claims, and knowing something about the geology and mode of occurrence of ores in the district and about the economics of engineering with respect to operation of a mine and its appurtenances, the surveyor is ready to make the actual location survey. The survey party usually is composed of a transitman and a chainman. If it is anticipated that there will be considerable clearing to be done in running the lines, one or two axmen may be added. The following items should be included in the equipment of the survey party:

Light mountain-type 1-min transit with complete vertical circle and an adjustable tripod;
Steel tape (500 ft) graduated at 5-ft intervals, the end 5 ft graduated to 0.1 ft or 0.01 ft;
Reel for 500-ft tape;
Steel tape (100 ft) graduated to 0.01 ft;
Pocket-size reel for 100-ft tape;
Self-winding, 5-ft steel pocket tape graduated to 0.01 ft on one side and to inches on the other side;
Detachable handle for end of 500-ft tape;
Two plumb bobs;
Level rod (12 ft) collapsible to 4 ft;
Axes;
Machete, or brush hook;
Finishing nails (8 penny);
Stiff white cards about 2 in. square;
Stakes (about 3 ft long and 1 in. by 2 in. in cross section);
Transit-type field book;
Standard field tables;
Location notice forms;
Brunton pocket transit; and
Cloth flagging and keel.

The transit should be light enough to carry around in the mountains. An adjustable tripod is a necessity for surveys in the mountains, where rocks, trees, and steep slopes complicate the setups. A complete vertical circle is far pref-

erable to a vertical arc, and the transit should be equipped with stadia hairs. Since a 4½-in.-diameter horizontal circle can be read about as accurately with a magnifying glass as can a larger circle, the smaller transit is preferable.

A method of traversing called the bearing traverse method is a favorite with many mineral surveyors. This method, which was developed by a group of mining engineers and mineral surveyors prior to 1900, is well adapted to land surveys in general and to mineral land surveys in particular. It yields more accurate results than any other method of traversing for the same amount of of time and labor expended. Since the bearing traverse method is not explained in any current text on surveying, it will be described briefly. A transit with two opposite verniers and graduated in quadrants is used. When a survey is started, the transit is oriented at the first station on true north as determined from the compass reading by correcting for magnetic declination, and bearings of lines are read directly from the A vernier. When a sight is taken to the next station, the bearing of the line is read directly from the A vernier and recorded. The transit is moved ahead with the upper motion clamped and the lower motion loose so as to retain this bearing on the upper plate. At the second station, the transit is oriented by backsighting with the telescope normal, and the lower motion is then clamped. Then the upper motion may be loosened, and the A vernier will give directly the bearing of any object sighted. The process is repeated at any other stations.

True bearings are ordinarily determined from a solar observation at some point, but occasionally by turning from a line of known bearing. Solar observations are much preferred over Polaris observations or other star observations because of the inconvenience entailed in being in the field at night and because of the extra lighting equipment which would be necessary. The solar observation usually consists of six observations on the sun with horizontal and vertical angles being read exactly the same as to any other shot in the traverse. The temperature is estimated for approximate correction to vertical angle. Ordinarily, the latitude of the place of observation is taken from a map of the state or of the district, the value being estimated to the nearest minute. Usually observations are taken every day at the beginning of an extended survey, and frequently in both the morning and the afternoon. If morning and afternoon observations do not agree, the assumed latitude may be in error. The six observations may be taken with the sun tangent in quadrants and the average of the observations calculated, but many engineers prefer to make the observations on the center of the sun and to calculate each shot individually. It is advantageous to take and to calculate observations on the center of the sun because the instrumentman can detect erroneous readings as they are taken, the computer can find instrumental errors due to variations in observations taken with the telescope normal or inverted, and the computer can discard "wild" readings instead of averaging all values. The assumed bearings, as determined by the original compass orientation, are adjusted to true bearings very easily, the correction ordinarily being less than a degree unless local attraction as a result of ore bodies is excessive.

There are two principal advantages of the bearing traverse method. First, the bearings of all lines are known immediately as the field work progresses—

without making and checking any calculations. This knowledge is particularly convenient when lines and ties of other claims are being retraced and when legal descriptions involving bearings are being written. Second, the precision obtained in turning the angles is as high as that obtained by doubling the angles, using any other method. This precision is due, of course, to the fact that any fractional part of an angle too small to be read on the transit (say, less than 30 sec) remains on the plates and is added to or subtracted from the next angle as it would be when an angle is doubled. Thus, there is no cumulative angular error. The method presumes that any eccentricity in the graduations of the horizontal plate of the transit is less than the error of setting or reading the vernier. To assure that the vernier setting has not been disturbed when the transit was moved forward, the vernier reading should be checked after the backsight is made. This check can be made more rapidly than the vernier could be set to zero, as would be done in running a deflection traverse. If the telescope is inverted in backsighting to orient, the bearing traverse method is essentially the same as that employed in the customary azimuth traverse method.[5] As in other methods of traversing, the magnetic compass bearing is read and recorded as a rough check on the transit angle. It is wise not to have the magnetic declination set off on the annular ring of the compass, as the compass and the transit bearings then will not be the same, and the observer is not likely to make two mistakes in readings which would still check.

The long tape, usually 500 ft long but occasionally longer or shorter, saves many an instrument setup and is much preferable to the usual 100-ft tape used in the flatlands. Of course, as in all other types of land surveys, the dimensions are given in horizontal measure. This theory is the only equitable and tenable principle in mining claims, where the surface measure usually is much longer than the horizontal. As an extreme example, the writer has worked on claims where it was necessary to measure nearly 2,500 ft to obtain the allowable 1,500-ft length of a claim—an average vertical angle of about 54°. Under these conditions it is entirely impractical, both in regard to time consumed and in accuracy attained, to use a short tape and to determine horizontal distances by "breaking tape." Vertical angles are read at the transit to the nail on the stake, and the tape is used to measure from the horizontal axis of the transit to the nail on the stake. The slope distances thus obtained are recorded and usually "reduced" as the survey progresses using cosine or secant functions from field tables so that the desired distance of 600 ft or 1,500 ft may be laid out. The chainman, as usual, carries the zero end of the tape. The instrumentman marks the distance to the horizontal axis of the transit with his fingers or with a pair of parallel pliers as the tape is pulled taut and then uses the 5-ft tape to measure the fractional distance from the 5-ft markers on the long tape.

Much of the measurement will be over rough ground and through timber where the tape will be at least partly supported. When a considerable part of the tape is swinging free, the instrumentman frequently corrects for the sag by applying more tension. Spring balances are practically never used and the

[5] "Surveying—Theory and Practice," by R. E. Davis and F. S. Foote, McGraw-Hill Book Co., Inc., New York, N. Y., 3d Ed., 1940, pp. 330–332.

correction for sag is not applied. The instrumentman can tell by the "feel" of the tape when he should take the measurement. If the precision required is not high, shots and measurements may be recorded only to the nearest 0.1 ft. However, it is usually necessary and more satisfactory to use the finishing nails to mark the exact points on the stakes and to measure distances to the nearest 0.01 ft. The 100-ft tape, graduated to 0.01 ft, is convenient and a little more precise for short shots. For a rough location survey where the terrain is difficult, a stadia shot might be used for determining a distance. This condition might be encountered where a ravine with considerable undergrowth in it is to be crossed, because of the difficulty of getting the tape across the ravine and clear of the growth. Inasmuch as stadia will not give the precision required for a patent survey, however, regulations of the Land Office prohibit its use.

The most convenient-sized stake is about 1 in. by 2 in. in cross section and about 3 ft long. This length stake will stand up high enough that grass and low brush will not have to be cleared, and will enable longer sights to be made. The 8-penny finishing nails, which mark the exact point on the top of the stake, will stick up about 1½ in. so that a white paper or card can be placed on it. This white paper shows up very clearly in any natural terrain, high lighting the nail and making it readily visible at ordinary distances. This nail is more accurate for sighting than a range pole, and is very much simpler and more accurate as a point for measuring.

Precision.—The precision required on the average location survey is a variable quantity depending on several conditions, the most important of which probably is the value or potential value of the land, which may be very difficult to determine at the time of the location survey. Some lode claims have been located where the entire value of the land probably would not pay the price of the survey. On the other hand, some locations have paid the owners at the rate of $1,000,000 per acre. The courts have been rather liberal in construing the intent of a locator. However, the position of the corner posts as actually established on the ground is the most conclusive proof of the intended position of a claim. Consequently, in any case where it is likely that there will be conflicts with other located ("unsurveyed") or patented ("surveyed") claims, the only safe procedure is to decide exactly where the new location should be and then place it there, maintaining a precision of, say, 1 in 1,000. This precision is not sufficient for a patent survey (as is explained later), but will do for most location surveys. In case it is probable that the owners will want to patent the claim rather soon, the surveyor is justified in making a more careful survey and in setting semipermanent points so that the patent survey will be mostly a matter of checking the original survey and setting permanent monuments. True bearings are usually determined by direct observations on the sun and would be correct to less than 5 min of angle; however, on some location surveys the magnetic declination is simply set off from the compass, in which case the bearings may be in error 30 min or more. Compass bearings are read as a check on the transit angles.

Monuments.—The monuments for a location survey ordinarily consist of posts about 4 ft long and at least 3 in. in diameter blazed on the side facing in

toward the claim and driven solidly into the ground. The blazed face is marked with keel to show the corner number and the name of the claim. When available—and this is more usual than not—a few stones are piled around the corner to help support the post; to help prevent live stock from scratching themselves on the post; and to mark the spot where the post was set, even though it may rot, fall over, and roll (or be carried) away. If timber is not available, a stone might be set and chiseled with the markings; but, since this is a rather slow procedure, posts are usually used for the locations. In case it would be impossible to set a post so that it would stay, such as in a road, a dump, a sliding talus slope, or a snowslide area, a witness corner should be set along one of the claim lines and plainly marked. In an attempt to be sure that the corners are of at least a semipermanent nature, some states have established laws requiring the posts to be of certain dimensions, a common requirement being a 4-in. square or 4-in.-diameter post at least 4 ft long, set 1 ft into the ground. An alternative of a stone mound 4 ft in diameter and 2 ft high is allowed in some states. All states, as well as the federal government, require that all the corners be marked by substantial posts. Some states (namely, Colorado and Wyoming) require that posts called "side centers" also be set at the center of the side lines (that is, 750 ft from either end for a 1,500-ft claim); one state, Oregon, requires that posts called "end centers" be set at the center of the end lines; and Arizona, North Dakota, and South Dakota require both side centers and end centers to be set in addition to the four corner posts. In most of these states the requirements of such a statute have been ruled by the courts to be directory and not mandatory and a violation does not invalidate the location.

Ties.—Some definite point or points, preferably the corners of the location, must be tied to a natural object or permanent monument so that the position of the claim is definitely fixed. On surveyed public lands where the section corners can be found, the tie might consist of the bearing and the length (course and distance) of the line from a corner of the location to the section corner or the quarter corner. The corner of the location which is thus tied to the section corner would probably be designated corner No. 1. In case there is a corner of some previously patented claim near by, a tie made to this patent corner would be entirely satisfactory. In some areas where there are no section corners or where the section corners cannot be found, United States mineral surveyors have been empowered to establish permanent monuments called United States mineral monuments (USMM) or United States locating monuments (USLM). These locating monuments or mineral monuments are usually a point such as a post, a large stone, or a chisel mark on a rock in place set on some prominent natural feature such as a knoll, a mountain peak, or a bench on a hillside. If the surveyor cannot readily find some recognized monument of one of these three types to which he can tie, he may, for the purposes of a location, tie to prominent natural features. One or more corners of the claim should be tied at least by course to some prominent or readily recognized mountain peak, to some road or stream junction, or to a well-known shaft, mouth of a tunnel, or permanent building. One such tie by course alone is quite poor. Consequently, accepted practice would be to tie to at least two such natural

features by courses from one corner of the claim, or to tie to one feature from at least two corners of the claim. This kind of tie, by giving a definite "fix" to the location of the claim, would thereby identify the tract to be described from any other tract.

As previously described, if the locator feels that it would be advantageous for him to take his ground in some shape other than a rectangle, this shape is entirely permissible. If there are more than four corners—as for a six-sided claim—corner monuments must be set at each angle of the claim.

Location Surveys for Placer Claims

If the mineral deposit is not in a vein such as quarrystone, oil shale, and the like, or if it is a loose deposit such as placer gravel containing gold, silver, tin, or commercial sand and gravel, it cannot be located as a lode claim. This type of deposit is open, however, to location as a placer claim under the mining laws [6] as amended by the Leasing Act [7] and by various Land Office regulations between 1920 and 1932. No stipulation is made by the federal government as to the dimensions of a discovery hole; but some states, such as Washington and Nevada, require a certain nominal expenditure in developing the claim. Courts have further ruled that, for location of a placer claim, the discovery must be an actual discovery of mineral values which can be recovered commercially.

Comparison of Gulch and Legal Subdivision Placers.—If the valuable placer deposits lie in a narrow, well-defined gulch, it is permissible to survey the 20 or more acres, depending on the number of joint locators, to locate only the valuable deposits. This so-called "gulch placer" results in taking long, narrow, irregular tracts of land, which leave very irregular and generally rather barren land in the public domain. Therefore, it is preferable—in fact, on surveyed public lands it is almost mandatory—to survey the placer claim in accordance with the generally recognized subdivision of a section, the smallest tract of which is a quarter of a quarter of a quarter of a section (containing, nominally, 10 acres). On unsurveyed (unsubdivided) public lands it is preferable to lay out the location with the cardinal directions and in a series of squares which are 10 acres in size so that these tracts could be fitted into a future extension of the public land surveys. Since about 1930, tracts as small as $2\frac{1}{2}$ acres have been subdivided out of the public domain for other purposes, such as governmental withdrawals, and there is no reason to believe that tracts of this size would not be acceptable for placer claims. However, there has not been any test case up to 1948.

If there are any known veins or lodes entering into or crossing a placer, these must be excepted from the area which is located and which may be patented. If it can be proved that a vein disappears and does not extend beyond the end line of the lode claim, it is permissible to locate a placer adjacent to this line. If one or more lode claims extend entirely across a placer location, the area will be divided into separate tracts, although it is necessary to have only one discovery for the one placer location, even though this location may be in several

[6] Act of Congress, May 10, 1872, Section 10; Revised Statutes of the United States, Section 2329, 2330, and 2331; 30 United States Code, Sections 35 and 36.

[7] "An Act to Promote the Mining of Coal, Phosphate, Oil, Oil Shale, Gas and Sodium on the Public Domain," Act of Congress, February, 25, 1920.

parts. If the mineral values are confined to a relatively narrow gulch, the same valuable mineral land may be more efficiently covered by one gulch placer of less than maximum size than by two legal subdivision placers of approximately 160 acres each. With larger-scale placer operations as compared to the hand operations of the past, taking up extra land of little or no mineral value in order to fill out 10-acre lots is less important. If 2½-acre tracts are to be permitted, the problem will be insignificant. Rules and regulations of the Land Office, as well as the decisions of various courts, differ as to whether or not the boundaries for the location of a legal subdivision placer claim need to be staked, and the surveyor must familiarize himself with local regulations on this point.

Methods of Survey.—When the placer claim lies entirely in one section which is well monumented, the survey problems are comparatively simple. However, the placer may lie in several sections. In the usual mountainous areas frequently the section corners not only are difficult to find in the first place, but are usually not placed very accurately or consistently if, indeed, at all. These difficulties necessitate rather extensive resurveys to relocate the section corners and to subdivide the sections into the desired tracts. A survey covering the several square miles involved of necessity must be a rather careful job to maintain sufficient precision. Otherwise, the methods of surveying a placer claim are no different from those for a lode claim. Corner posts would be of the same type as that described for lode claims, with the markings indicating the corner number and name of the placer. In the case of a legal subdivision placer, the tracts are described by their position in the section. If the section corners are in place, no other ties are necessary for the identification of the tract. In case some corner of the placer claim does not coincide with a section corner or USMM, ties would be run on the ground, as is done with lode claims.

Location Surveys for Mill and Tunnel Sites

The mill site, which is the third standard type of mineral land claim, is probably the simplest to survey. It is limited to a maximum area of 5 acres; the shape is not restricted, but it should form a reasonably compact or regular tract. Of course, more than one mill site can be taken up by one locator provided that a need for and use of the land can be shown. As previously stated, a 5-acre mill site may be taken up with a lode claim. The need for this type of mill site has nearly disappeared with the advent of present transportation and a few large ore treatment plants. A mill site may also be located and patented as a site for an actual ore reduction plant. The ground which a mill site covers should contain no known mineral deposit, which usually precludes the location of a mill site on the end line of a lode claim. The problem in surveying consists in laying out the tract and setting a monument at each corner of it, as is done in the survey of a lode claim location. It is permissible to have the tract composed of two or more near-by parcels of land.

There is one other type of location on mineral land, on which, however, the government will not issue a patent. This is a tunnel site. The tunnel site may claim 3,000 ft in advance of the face of and along the line of a tunnel, together

with 1,500 ft on either side of the tunnel. The filing of this location protects an investor who is developing an area with a deep cross-cut tunnel and grants him the exclusive prospecting rights in the 3,000-ft-square area. If any new lodes are discovered as the tunnel progresses, they would then be located in the same manner as an ordinary lode claim with 600-ft by 1,500-ft dimensions. Lodes known to exist prior to a tunnel location do not restrict location at the time they are cut by the tunnel, but are open to location in the usual manner. If desirable, a square "dump site" 125 ft on either side of and 250 ft below the mouth of the tunnel may also be staked out, if this area is on nonmineral land. Staking a dump site consists of monumenting the four corners; staking a tunnel site consists of monumenting the four corners and also the end of the tunnel center line 3,000 ft from the mouth, or from the face, as the case may be.

References or ties to some permanent monument or natural object are the same for mill site or tunnel site locations as they are for lode locations. A mill site located in conjunction with a lode claim should give a tie by course and distance between a corner of the lode and a corner of the mill site.

Completing and Maintaining the Location

The surveyor, after staking out the lode or placer claim, should post a notice of discovery and location at the point of discovery and should advise his client as to the validity of the discovery hole. The surveyor should fill in a location certificate form for his client to file in the office of the county clerk. This certificate should include the name or names of the locators, the date of location, and the dimensions of the claim, including such a description as will positively identify the intended tract of ground. The steps necessary for completion of a mill site or a tunnel site location are essentially the same, with the obvious exception that a discovery does not have to be made.

After initiating a mineral land claim, the locator has certain rights to the land he embraces as to the methods of mining or development. He may retain these rights for himself, or he may lease or sell them to another. However, in order that the location may not become void and the land thereby "jumpable" —that is, open to location by another—the locator or his heirs, assigns, or legal representative must do certain development work (usually called assessment work) which amounts to $100 worth of labor or improvements on each claim during each year. For a contiguous group of claims all this assessment work may be done in one place, as long as it can be shown definitely that such work is a benefit to each of the individual claims.

Title to the land claimed by a location remains vested in the federal government until such time as a patent is issued. Consequently, the locator pays no taxes on any locations. Assessment work is not required on a mill site or a tunnel site. There have been a few depression years and war years during which Congress suspended the requirement for annual assessment work on lode and placer claims.

The matter of assessment work may seem to have little to do with surveying, but in actual practice the surveyor must be able to judge whether ample assessment work has been performed to enable his client to establish and to maintain a clear title to the claim. The United States mineral surveyor must be con-

versant with the costs of performing various types of mining work, inasmuch as he must swear under oath that at least $500 worth of mining improvements have been performed toward the development of each claim on which a patent is sought.

Patent Surveys

As soon as a locator has determined that his claim or claims have sufficient value to warrant his obtaining a fee simple from the government, he applies for a patent. At this point, the surveyor reenters the picture to survey the land completely and carefully so that the government may adequately note the position of (segregate) and issue title (patent) to the tract of land. This time the survey is made under official order by a duly qualified and appointed United States mineral surveyor.

Mineral Surveyors.—Mineral surveyors are commissioned on application after proof of technical qualification, general reputation, and fitness for the position. The appointment is made by the United States office cadastral engineer through the public survey office for the particular land district or state to which the application is made. The mineral land surveyor, on receiving his commission, must post an official bond in the sum of $5,000 before he may make official mineral surveys.

In 1948 there were 118 men commissioned as mineral surveyors in the United States and Alaska. These men follow the regulations and are under the supervision of the Public Survey Offices, although they are not under civil service nor do they receive any compensation whatever from the federal government. Prior to 1925, the surveyors were called United States deputy mineral surveyors and were under the office of the surveyor general for each of the so-called public land states. The appointment to the position of surveyor general for a state was political, so that the term of office was usually short, and the position was frequently held by some one who might know very little about mineral surveys. Prior to the issuance of the first manual of instructions [8] the survey procedure was often different in different states, and even different in one state from time to time. These differences in procedure would be occasioned when the commissioner of the General Land Office issued special instructions by letter to a surveyor general or to a deputy mineral surveyor to cover some special condition, which might be treated differently if a similar case arose elsewhere. Occasionally much confusion enused. In 1895 the manual of instructions was issued by the commissioner of the General Land Office as an average of the practices in various states. With each subsequent reissue of the manual, variations in survey practice have decreased until at present, with only slight exceptions in Alaska, all surveys conform to the standards of the 1947 manual.[2] Although 118 mineral surveyors in the United States and Alaska may seem a small number of men to carry on the mineral survey field work, especially when it is realized that more than 300 surveyors held commissions as deputy mineral surveyors for Colorado alone in 1900 as compared to 22 mineral surveyors in 1948, still there are only 40 commissioned surveyors

[8] "Manual of Instructions for the Survey of the Mineral Lands of the United States," General Land Office, U. S. Dept. of the Interior, U. S. Govt. Printing Office, Washington, D. C., 1895.

of all the public lands of the United States. In other words, there are at present more than twice as many mineral land surveyors as there are public land surveyors, indicating the importance of mineral land survey work.

Patent Application.—The procedure for obtaining a patent to mineral land is much different from and more complicated than the process of obtaining a patent to any other of the public lands. This situation derives principally from the fact that for agricultural or similar lands entry is made on a quarter section which has already been surveyed, monumented, and platted according to Land Office methods, whereas for mineral lands a survey of the particular tract must be made, monuments must be set, and a plat must be made so that conflicts can be shown and computed before the local land office can issue a patent.

The owner or owners of a location or locations should first employ a United States mineral surveyor of his or their own choosing to make the patent survey. Before the claimant applies for an order for an official survey, the surveyor, if he did not make the original location survey himself, should make a careful survey to ascertain that the stakes on the ground are where they belong, that the claim is of the proper size, that the desired ground is covered, and that the location certificate properly describes the claim. In a great many cases it is necessary to amend the location by filing an amended location certificate to accomplish these objectives. The official survey is bound by the location certificate, only slight variations being permitted, so that the major changes usually required by poor certificates can be made only with great difficulty, if at all. The survey made for the purpose of ascertaining the correctness of the location, and for obtaining the data for the amended location certificate, is conducted with the same care as the patent survey itself. Indeed, it is a necessary part of a patent survey, even though it is made before the order for official survey is issued. If an irremediable mistake between the location as staked and the location as described in the certificate is discovered after the official survey order is issued, it will be necessary to amend the location certificate, cancel the old survey order and number, and obtain a new order and survey number. The mineral surveyor will have to certify that $500 worth of labor and improvements have been made for each claim for which patent is applied.

The claimant, who must be a citizen of the United States, or a person who has declared his intention to become a citizen, or a corporation chartered in the United States, applies for an order for survey. The application must be accompanied by certified copies of the location certificates or of the amended location certificates, as the case may be, obtained from the county recorder. The public survey office estimates the cost of office work on the survey; and, when the claimant has deposited this amount, the order for mineral survey is sent to the deputy selected by the claimant, together with copies of the certified copies of the location certificates.

Making the Survey.—The mineral surveyor proceeds with reasonable diligence to make the official survey, progress depending somewhat on the season of the year and on similar factors.

Ties to Monuments.—In beginning the patent survey, whether it be the relocation survey or the true patent survey, the surveyor should determine first whether the claim is on surveyed or on unsurveyed land. If the survey is

on surveyed land in good standing, he must find the nearest section corner to the claim, provided one exists within 2 miles. In the mountains this frequently is difficult. The section corners in most of the mineral districts are stones or, occasionally, posts. If there have been other mineral patent surveys in the area, the field notes of these claims will show what monument was tied to and may assist the surveyor in his search. In the event that a public survey corner cannot be located within 2 miles of the claim being surveyed, it is possible that a USMM or a USLM has been established previously in the vicinity. A monument of this type may also be used to tie a mineral claim to if it is within 2 miles from the claim. If the mineral surveyor cannot find a section corner, a USLM, or a USMM within 2 miles of the claim, he is empowered to establish a USMM according to the procedure set forth in the manual of instructions.[9] In times past, the mineral monuments or locating monuments have been designated by letters in the greek alphabet, by Roman numerals, by the names of the mining district, or by the names or the survey numbers of the claim being surveyed when the monument was established. There are (1948) more than 700 monuments of this type in Colorado alone. The connecting tie from corner No. 1 of the claim to the section corner or USMM is supposed to be run directly on the ground, or the connecting traverse is supposed to be described in the field notes which the surveyor submits to the public survey office.. Nevertheless, common practice is to traverse or to triangulate the tie, and then to calculate the direct line and report its bearing and length. The tie may be calculated through a tie from some near-by claim if that tie has been verified.

Ties to Conflicting Claims.—The surveyor should next find the corners of any conflicting claims. There are two prerequisites in the search for these corners: First, a map showing the district with the patented claims platted on it, available from the public survey office of the state as a "segregated sheet" or published by a mineral surveyor in the district; and, second, an abstract of the field notes of the patented claims likely to be in conflict with or near the claim to be surveyed, obtained by consulting the original field notes on file in the public survey office or the notes published by some individuals.[10] In some of the larger districts, index books have been published giving the township and sections in which each patented claim is located.[11] Such an index book may also help to locate any conflicting or near-by claim.

Recovery of Corners—Starting at the discovery hole the surveyor proceeds to locate any patent corners which are on lines of existing patented claims conflicting with the proposed claim. If there are no conflicts, the problem is simple. The claim is laid out in conformity with the location stakes and the location certificate; monuments are set and marked at the corners of the claim; a tie is run to a section corner or other public land monument; and, under favorable circumstances, the survey may be completed in a day or two. However, in the old established districts, with many conflicts being the usual situa-

[9] "Manual of Instructions for the Survey of the Mineral Lands of the United States," Bureau of Land Management, U. S. Dept. of the Interior, U. S. Govt. Printing Office, Washington, D. C., 1947, pp. 451–452.

[10] "Abstract of Survey Field Notes in Cripple Creek District, Colo.," by Fred J. McNair, Denver, Colo., 1897.

[11] "Cain's Index of Mining Claims in Lake County, Colo.," by William P. Cain, Leadville Pub. & Printing Co., Leadville, Colo., 1906.

tion, the surveyor may spend a week or more just looking for and recovering old corners.

As has been the case with all land surveys from time immemorial, the corners of early mining claims have been poorly monumented according to modern standards. In addition to the usual ravages of the elements, inasmuch as most mining districts are located in mountainous areas, the monuments generally fall on slopes where they are subject to the general subsidence of the land form, the destructive forces of snowslides, damage from rolling rocks, burial by dumps from mines, and other destructive factors not common to the average land survey monuments. The mortality among these patent corners is high. Because there are practically never any fence lines along the claim boundaries, and because the timber which may have existed and been blazed at the time of the survey has usually been cut for use in the mines or has been swept by forest fire either accidentally or purposely, the problem of retracing the boundaries and recovering the monuments is a difficult one indeed.

Most of the early corners were wood posts. They were required to be a minimum of 4 in. square and 4 ft long, squared at each end and set at least 1 ft deep in a dug hole (not pointed and driven into the ground, as was occasionally done). The proper markings were carved into the wood, some deputies exhibiting great artistry with a knife. At the time these monuments seemed very satisfactory. The posts stood up above the surrounding land and, particularly above timber line or on open land, were prominent and easy to find. With the usual mound of rocks around it, the post was reasonably stable and would last for quite a few years. If the deputy used care in selecting the post and chose a cedar, or "pitch" pine, or other rot-resistant wood rather than an aspen or cottonwood, and if the post was hewn rather than sawed, its lifetime might be doubled or tripled. Probably not more than 10% of the original post corners last more than 30 years without being repaired or replaced.

Some of the better mineral surveyors in the early days, recognizing the importance of monumenting the corners in a more permanent manner, would set a stone corner if they could do so without too much additional labor. These stone corners are, for the most part, still extant. Many of the earlier stone corners, because they were too small or were poorly set, have subsequently been destroyed. Some of the stones were of poor quality; and, if the surveyor finds that the chisel markings on these stones are entirely obliterated, he may have some reasonable doubt as to the authenticity of the monument. For a good monument a hard, weather-resistant rock at least 24 in. long and of at least 6-in. minimum thickness in transverse section (say, as large as one man can handle or roll into place) should be selected, set with two thirds of its mass underground, and firmly braced in place. In addition, it should have an earth backfill thoroughly tamped around it and should be surrounded by a pile of rocks. Such a monument will withstand all but the most extreme of natural destructive agencies.

It is not especially unusual for a corner in the mountains to fall on a large boulder or on solid rock. In this case, the chisel markings, if made properly, will last almost indefinitely. Occasionally, a tree may be at the corner point; if so, it will be blazed and properly scribed, with a nail placed at the corner

point. A suitable tree, adequately marked, makes a good permanent monument. In rare cases, cut stone, heavy mine rail, light railroad rail, or concrete posts have been used at the corners. Since 1930 a very few iron pipes with flaring base, concrete core, and brass or aluminum cap for marking with steel dies, similar to the type adopted for public land survey monuments, have been used. These have been highly satisfactory except in the rare cases where minerals in the soil or water may attack the iron pipe. Therefore, despite the additional expense for installation, pipes will probably be used to a large extent in the future. However, all these last few materials are usually relatively expensive and difficult to transport to the site. It is difficult to impress the average owner with the necessity of establishing monuments that will be as permanent as possible, although more expensive.

From the abstract of the original field notes the surveyor ascertains that the corner he is looking for consists of a certain material. If there are several conflicting claims, there may be several references from one survey to another which may help to show just where a corner may have been previously found, and in what condition. Of course, the corners may have been reset or replaced and may possibly be of some other material than the original, although the latter case is unusual. As he makes his search, the surveyor should have a full description of any local bearings such as bearing trees or bearing rocks. This information is particularly important for post corners, since in most cases these local bearings may be the only evidence left as to the position of the corner.

Historical Considerations.—After one corner has been identified to start from, it is frequently possible to set the transit over this point and to rerun the old lines and ties described in the abstract of field notes, thus recovering other necessary corners. However, historical considerations as described hereinafter frequently complicate the seemingly simple operation of retracing old lines.

As previously described, the variations in individual instructions caused much confusion in the very early patent surveys, as each surveyor general issued his own instructions for the conduct of each survey. In addition, in each mining district in a state, each claim had its own mineral survey number, beginning with No. 37 (presumably on the hypothesis that there were already thirty-six sections or tracts in a township). Between 1879 and 1890 all these mining districts were coordinated in each state, and since then the numbers in each state have been assigned consecutively. Colorado, with the greatest number, has 20,709 survey orders or a total of 27,940 including the subtotals from the seven separate districts. Montana, with the second highest number, has 10,802 survey orders. Until about 1883, in general, only single claims were patented; after that time two or more contiguous claims could be patented as a group and with only one survey order number.

The very earliest patented claims were cut off where they conflicted with another claim as in the case with the Beverly lode (Fig. 2(*a*))—resulting in an odd-shaped tract. This practice was soon discontinued, because any error in the survey or in the setting of the monuments might leave a small fraction of unpatented ground between the claims. For the next few years, one lode was prohibited from lapping across another prior lode as is done by the Horseshoe

lode location to acquire area A (Fig. 2(*b*)). Between the years 1891 and 1900 a ruling required that a claim such as the Garnet lode (Fig. 2(*c*)), which conflicted with a prior lode, must be cut off at the point where its lode line intersected the prior lode. This practice caused many small unpatentable fractions of land to be created. Fig. 2(*d*) shows a special case of this same ruling where, by agreement of the two owners, the Ben lode could be extended to obtain a patent to additional tract B. Since 1900, the claims have been permitted to extend into one another. Of course, if no vacant ground is covered, there is no advantage in extending a claim the full 1,500 ft in length or the full 600 ft

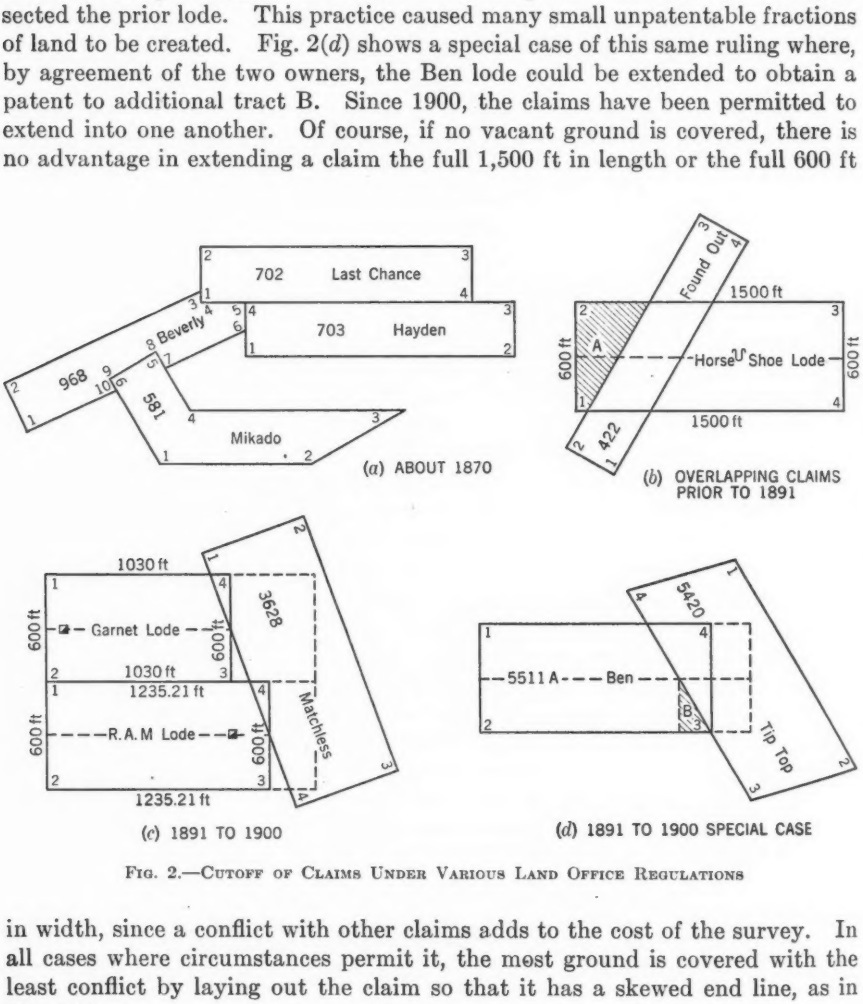

FIG. 2.—CUTOFF OF CLAIMS UNDER VARIOUS LAND OFFICE REGULATIONS

in width, since a conflict with other claims adds to the cost of the survey. In all cases where circumstances permit it, the most ground is covered with the least conflict by laying out the claim so that it has a skewed end line, as in Fig. 1(*b*).

Originally, claim corners were numbered either clockwise or counterclockwise. In some groups of claims, a common corner might be numbered—for each claim. In Colorado during the period from 1886 to 1892, the corners were numbered consecutively through the claims in the group. This system of numbering is convenient because a person finding corner No. 18 knows immediately where he is with respect to the entire group, instead of knowing only that if he has found a corner No. 2 of some one of perhaps a dozen claims in a group. From 1892 to 1925 the corners were simply numbered in rotation

from 1 to 4 (or 6, or 8, depending on the number of sides) around each individual claim, which led to much confusion. Since 1925, the corner nearest the section corner, or USMM, must be No. 1 and the mineral surveyor has been required to put the initial or enough of the name of the claim on each corner to permit its identification.

The main cause of confusion and difficulty in retracing old surveys is the so-called "section-tie" ruling which was in effect from June, 1899, to August, 1904. This Land Office regulation required that existing claims were to be tied to at the place where the approved field notes described them, regardless of where any existing corners might place the claims. At least on paper, this ruling would appear to simplify the surveying and to insure the most economical coverage of the ground. However, this regulation would be comparable to a ruling which would require that henceforth all section corners are to be considered as located where the original subdivision survey notes place them, rather than where they actually may be found on the ground. Such a ruling is contrary to all court decisions, since the position of existing monuments on the ground is considered the best evidence of the true and desired location of a tract of land. The largest differences between the described position and the monumented position of a claim occurred when there was an error in the tie to the section corner or USLM, hence the name "section-tie" ruling. These errors varied from negligible amounts up to several thousand feet, the latter usually occurring where a section corner was not properly identified. Of course, differences in the claim were also caused by errors or mistakes in the survey of the boundaries themselves. The status of all claims found to have erroneous descriptions, or to be in conflict with claims which did have, was very uncertain during the period that the "section-tie" ruling was in effect. Consequently, many of the old claims with imperfect descriptions were "repatented" even though the monuments of both surveys fell at the same place. Similarly, many of the claims which were patented during this period and which were adjacent to or conflicted with claims having erroneous descriptions had to be resurveyed and "repatented" after the repeal of the Land Office ruling, because during this period, if the claims were to occupy a certain vacant parcel of ground, the erroneous ties and lines of the adjacent surveys had to be called correct and reconsideration of claims proceeded on that basis. In searching for old corners and in making the present survey and description, the mineral surveyor must keep all the foregoing facts and approximate times in mind.

In the earliest days of mineral land surveys many of the deputies used the the old link chain for making measurements. As these chains were used, and as the links wore, the measurements gradually became longer and some of the claims were thereby made too long. During the early 1890's, the chain was superseded by steel tapes, and thereafter errors from this source disappeared. On the other hand, many of the early claims are too short because the deputy surveyor had been making public land subdivision surveys and was familiar with "breaking chain." He would badly misjudge the mountain slopes and it is not unusual for the engineer on a resurvey to find a claim described as 1,500 ft long from as much as 15 ft to 30 ft short.

Recently it has been determined that general earth movements after many of the claims were surveyed (in some cases as much as 75 years ago) have caused movements of entire claims or of one corner of a claim with respect to the other corners. This movement is, of course, most noticeable in the steeper mountains, where the shape of the slope may seem intact; and, indeed, the individual corners may appear to be very firmly set in their original positions. Nevertheless, careful surveys made at intervals of 10 years or more plainly show gradual movement and settling of an entire mountain side. In one particular case where a rather valuable claim in Colorado was surveyed for patent in 1876, side-center monuments were set on a claim, one of the side-line points falling on a ledge of solid rock, and therefore being marked by an *X* chiseled on the ledge. The claim lies along the face of a steep mountain and on a slope composed mostly of slide rock. All the corners have apparently remained "in place" continuously. In 1900, this claim was resurveyed by another surveyor during the process of surveying an adjacent claim, and it was discovered that the side line departed from a straight line by about 3 ft at the end—that is, there was an angle of about 0° 05′ in the claim at the side center as shown in Fig. 3. By 1928, the movement had become so large that several apparent

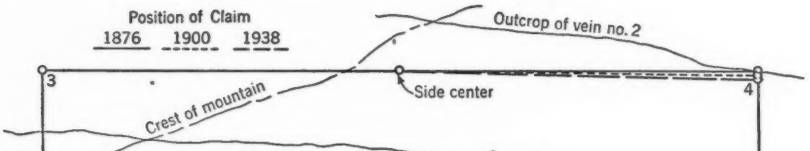

FIG. 3.—MOVEMENT OF MONUMENTS DUE TO EARTH SUBSIDENCE

fractions amounting to as much as 5 ft or 6 ft had appeared, the movement being such as to cause the apparent apex of the vein to cross the end line rather than the side line; and these fractions were then patented to insure complete coverage of the vein. In this case the title to more than $1,000,000 worth of ore was affected. In 1938, the surveyor who had made the survey of 1900, and others at various intervals since that time, had occasion to make a rather extensive survey of all the claims in the area. By rerunning the lines as they existed in 1900, he was able to show that the fractions between the original claims were probably nonexistent in the surveys of 1876.

Another instance of this earth movement is a claim which has one end line on top of a mountain ridge and which extends 1,500 ft down a slope consisting of slide rock inclined about 35°. One of the monuments on the lower end line is a large stone set in a niche on a point of solid rock. To all appearances this stone is as stable and permanent as possible. This claim, as determined by several independent measurements, was 1,500 ft long when it was patented in 1901. By 1942 the monuments had moved apart, the lower point of solid rock apparently settling downhill, so that the length of the claim between monuments was 1,504 ft. The examples just cited are believed to be unusual in the

United States because of the time interval between unquestionable observations and the magnitude of the movement. Some studies have been made of the movement of rock glaciers (great masses of broken rock which "flow" in a manner similar to the familiar ice glacier). The report of these studies,[12] some of which are as yet unpublished, may throw some light on the apparent errors in some old mineral surveys which, although made by reputable surveyors, show unaccountable errors.

Earth movements are also caused by known active faults or by settling due to mining operations. All instances of movements of the monuments pose very interesting and complex problems of title in which engineers and land surveyors are vitally concerned, particularly in court cases.

Precision.—On all mineral land surveys made since 1872, the required precision, measured as an error of closure around the claim, is 1 in 2,000. This error "must not be such as to make the claim exceed the statutory limit"—that is, if one side line is 1,500 ft long, the other must be between 1,499.25 ft and 1,500.00 ft long. However, even in difficult terrain, the mineral surveyor can and usually does, maintain a precision of about 1 in 5,000. The angular variation allowed is 2 min.

Prior to 1872, the magnetic compass was used for determining the bearings of lines on some mineral land surveys. Since 1872, use of the compass has been prohibited and direct solar observation with a transit has been required. For many years Polaris observations were not permissible, despite the fact that they can be more precise than a solar observation. At present (1948) either a solar or a Polaris observation is acceptable, but, as previously stated, most mineral surveyors prefer the solar observation. Ordinarily six observations with three direct and three reversed will produce results which will agree within 2 min of angle. True bearings for the lines of many of the first surveys in any given area were carelessly determined, perhaps because compass values alone were read. However, as the land has become more valuable and other surveys have been made with greater care, the determinations have been quite satisfactory.

As in all surveying field work, each surveyor has his own technique and procedure—some use the deflection method, some the azimuth method; some double the angles, others do not; and some orient by the method of plate reversals, some by plunge reversals. Probably the method used by more mineral surveyors than any other is the bearing traverse method previously mentioned. In all cases where the ground is at all irregular or steep a long tape (usually 500 ft long) is used. Vertical angles are read, and slope measurements are made and reduced to give true horizontal distances. Stadia measurements are not permitted on patent surveys. Occasionally the terrain lends itself to a triangulation network, particularly if several claims are included in one survey order so that a large area is covered. In this case the lines between the triangulation points might span some of the rough or heavily timbered terrain and only short traverse lines would have to be run to the discovery holes or to the position for corners. Conditions would vary depending on the extent of the triangulation

[12] "Rock Glaciers in the Colorado Front Range," by Ronald L. Ives, *Bulletin*, Geological Soc. of America, Vol. 51, July–December, 1940, p. 1271.

network, but rarely would the angles be repeated more than three times with a 1-min transit. The base line for such a system would probably be double chained, and a spring balance and temperature corrections might be used to obtain greater accuracy for an extensive system. Since aerial photographs are available for most of the mineralized area in the United States and Alaska, mineral surveyors will become more familiar with their use. Particularly where several locations are embraced in one survey, the aerial photo gives an over-all view of the area and is an invaluable reconnaissance aid to the surveyor. Essentially, it gives him a map of the area before he starts into the field so that he can select, for his traverse lines and triangulation stations, positions which will avoid heavy clearing and difficult field work.

Monuments for the corner points would be composed of rock in place, iron pipes, set stones, or posts, in that order of preference, as indicated previously. At each corner one or more bearing trees or bearing rocks should be located with respect to the corner wherever this is possible. Such local bearings preferably should be within 50 ft of the corner. If the position of the corner falls in a road, stream, loose bank, or other place where a permanent monument could not be established or maintained, a witness corner would be set on one of the lines of the survey and as near the true corner as possible. Of course, complete descriptions of the corner monuments, bearing objects, or witness corners would be included in the official field notes to insure identification of the true corner at any future time.

Field Notes.—The surveyor is required to run a tie between the nearest public survey monument and the corner of each of the claims he is patenting which is nearest to the monument. In cases where the claim being surveyed conflicts or lies near previously surveyed claims, the surveyor in his official field notes reports how he identified the corners which he found on these old claims and what condition they were in or how he may have reset or re-marked them. He is also required by the Land Office to run a connecting line from some corner of the claim he is surveying to a corner of each conflicting surveyed claim, and also to a corner of any conflicting unsurveyed locations which will exclude area from his claim. In addition, more recently, connections have been required to any near-by claims even though they may not conflict. Also, for the claims which do conflict, the surveyor's field notes must report the distance along the line of the present survey to the point of intersection with the line of the old survey and the course and the distance from that point along the line of the surveyed claim to one or the other of the corners. If any corner of these conflicting claims falls within the claim being surveyed, the course and the distance from the point of intersection of the lines to this particular corner should be given. Of course, to give the bearing and the length along a line, monuments at the ends of the line must be located. In the event that all the corners required for fixing the positions of conflicting lines cannot be recovered, the mineral surveyor must exercise considerable judgment based on past experience and on the evidence at hand as to the probable location of the conflicting claim. The record position of the claim may be adopted as a basis for defining the conflict. In any case, the field notes must give a complete de-

scription of the evidence at the time of the survey and the reasons for the adopted values.

Joint Surveys.—In addition to reporting the condition of the corners which are found, the mineral surveyor must report in his official field notes any differences which he finds in the approved field notes of the conflicting claims. In earlier days whenever a deputy reported another claim in error, the deputy who made the conflicting survey was required to resurvey the disputed area and to file amended notes concerning the errors reported. In case the deputy who surveyed the first claim denied that there was an error, a joint survey was ordered by the surveyor general to be made by the two deputy surveyors jointly. The deputy found in error paid all expenses and $10 per day to the surveyor found to be substantially correct. This practice generally produced good results because no reputable surveyor wants to be shown that he has made a mistake. However, the system became impractical and was abandoned about 1925, because so many of the earlier surveyors had died that joint surveys were usually no longer possible. At the present time (1948), errors are reported and notations to that effect are made on the original official plat by the public survey office.

Placer and Mill Site Surveys.—The procedures for the survey of placer or mill site claims are essentially the same as have been described for lode claims. If a lode and a mill site are both included in the same survey, the lode claim survey number is followed by the letter A and the mill site number is followed by the letter B. A tie must be run from corner No. 1 of the mill site to some corner of the lode and also to a public survey or a mineral monument. If a placer claim is separated by known lodes into two or more tracts, each of these sections should have a separate series of corner numbers. To avoid confusion some surveyors call the corners in one tract Nos. 1, 2, 3, and so forth, and those in another tract A, B, C, and so forth. When a placer conforms exactly to the public land system, no survey is necessary to segregate the area, and a patent may be issued directly. The placer then has a mineral entry number similar to the familiar homestead entry numbers. If there is any doubt as to the exact positions of the public survey monuments, as there is in many mountainous regions, the applicant will be wise to apply for an official mineral survey, to be sure his corners cover the land he wants. It is entirely possible that one claimant might have contiguous lodes, together with mill sites and placers, all to be surveyed under one order of survey.

Improvements.—At some time during the progress of the survey the mineral surveyor must locate all mining and other improvements. On lode claims these improvements preferably will be located by course and distance from some corner of the claim, although they may be tied to some point on the lode line—usually the center line—of the claim. A total of $500 of improvements since the date of discovery must be shown to obtain a patent, and the mineral surveyor must certify to this value. The only improvements whose value applies toward the necessary $500 are mining expenditures made by the claimant since the date of discovery. The discovery hole is the first such improvement. Except for the discovery hole, the improvements may be outside the

claim if such improvement actually benefits the claim—for example, a drainage tunnel for deep development of the claim. Where more than one claim is being patented in a group, all the development work may be done in one place, with each claim being credited with an equal share of the total value of the improvement if all these claims are in a common ownership. Other improvements such as roads, buildings, and other structures ordinarily not classified as mining expenditures, as well as any topographic features such as streams, roads, or cliffs, should be noted since all these may help to identify the position of the claim. On a mill site taken up in conjunction with a lode, no expenditures are required beyond the $500 required on the lode. On a mill site taken up as a site for a reduction plant, the structures should be located.

Since September, 1882, a descriptive report has been required[13] on all placer claims, including type of soil, type of timber, size and position of streams, improvements by the claimant, and many other items. The field work of locating the various improvements and topographic features is the same as for any ordinary mine survey or topographic survey and need not be described in detail.

Occasionally a claimant may attempt to obtain a patent when he has not performed sufficient work on his claim or claims. In this case the mineral surveyor must remember his code of ethics and that he is under oath; he must refuse to make his official statement on the value of the improvements until the work has been done. Honest differences between the actual cost and the apparent value of some improvements occasionally arise. Each of these cases must be considered by the mineral surveyor on its own merits.

The surveyor prepares a plat of the survey on a scale of 200 ft to the inch, writes up the field notes, makes his area calculations by the method of double meridian distances, and sends copies of all these to the public survey office for the state. The form for each of these is specified, the details being given in the manual of instructions.[14] If everything is in order and the notes and calculations are correct, the cadastral engineer approves the survey. If a mistake is found, the notes are returned for correction. Occasionally, through some irregularity, an amended survey of the area may be ordered by the Director of the Bureau of Land Management. Special instructions as to the object and methods of the amended survey are issued to the mineral surveyor. Every place the survey number of this amended survey appears, whether on the monuments in the field or in the notes, the abbreviation "Am." is used after the number.

The services of the mineral surveyor are supposed, by law, to be completed at this point. However, there are many legal forms to be filled in and land descriptions to be written. If no other claimant files an adverse claim to the area embraced in the survey within 60 days, the patent is then issued to the claimant on his payment of $5 for each acre or fractional part of an acre for lode claim or mill site areas, or of $2.50 for each acre or part of an acre for placer acreage.

[13] "49 Land Decisions," General Land Office, U. S. Dept. of the Interior, Washington, D. C., April 11, 1922, Vol. 58, Paragraph 167.

[14] "Manual of Instructions for the Survey of the Mineral Lands of the United States," Bureau of Land Management, U. S. Dept. of the Interior, U. S. Govt. Printing Office, Washington, D. C., 1947, pp. 453–458.

Conclusion

Nowhere (as of 1948) in literature is there any complete presentation of the history, methods, and instructions for the survey of tracts of mineral land—although this land is the only type of public domain patentable in the United States and although all land is rising in value. Because a land surveyor, civil engineer, or mining engineer may have to resurvey or subdivide these tracts of mineral land, which are numerous, particularly throughout the western part of the United States, this paper has presented the general features of mineral land surveying. The day of the small prospector is about gone; and, as the individual claims are consolidated into larger groups under one management, it will be increasingly important to survey and patent the small, irregular, unpatented fractions of land left between the original claims. This important work will necessitate a thorough knowledge of the historical conditions at the times of those earlier surveys, as herein presented. In new mineralized districts, yet to be found, the claims no doubt will be laid out in more regular form so as to avoid all the conflicts and exclusions of the older system. Moreover, the importance of monumenting the tracts of land as permanently as possible cannot be overemphasized, since the engineer retracing old boundaries finds the greatest difficulty and spends the most time with this phase of the work. This paper, in presenting some of the techniques of the past, should help the engineer to save time on resurveys involving tracts of mineral land.

Acknowledgments

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

PANAMA CANAL—THE SEA-LEVEL PROJECT
A SYMPOSIUM

Discussion

BY GEORGE B. MASSEY, WILLIAM ALLAN, AND BORIS A. BAKHMETEFF

GEORGE B. MASSEY,[102] M. ASCE.—Three of the papers in this Symposium are of especial interest to the writer—the fourth, fifth, and ninth.

Tidal Currents.—In India, the large irrigation canals must have a certain water velocity between very narrow limits so that silt will not be deposited or will not be picked up from the bed of the channel and carried away. To arrive at the necessary hydraulic radius some very unusual cross sections for canals are developed.

The problem in the sea-level canal at Panama is to provide a free, open channel for ships and, as much as possible, to prevent the tidal conditions from causing an undesired current. With that idea in view, consideration should be given to the advantages and disadvantages of submerged natural stone weirs left in place athwart the channel at intervals of 1,000 ft, with top elevation at −42 ft, a top width of 10 ft, and 45° side slopes. It is probable that the reaction of a passing ship to these weirs would scarcely be noticed; and, as a retarding agency to the velocity of the current, they should have a considerable effect.

If these weirs were left in place when the canal was first used, the result would be that less rock would have to be excavated before the canal was opened. If found ineffective, they could be removed with the canal in use.

Ships would have all the advantage of a 60-ft depth of channel, with the further advantage of a decreased velocity.

Ship Performance in Restricted Channels.—In connection with the many tests in the towing basin, it would be interesting to know if any were made with a central dike having a top elevation of −42 ft, a width of 10 ft, and side slopes of 45°, to determine whether this factor would tend to a favorable modification

NOTE.—This Symposium was published in April, 1948, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: June, 1948, by Hans Kramer, and Philip G. Nichols; September, 1948, by George B. Pillsbury, Kenneth S. M. Davidson, Joel D. Justin, W. H. McAlpine, W. E. R. Covell, William Herbert Hobbs, Hibbert Hill, Kenneth C. Reynolds, Gregory P. Tschebotarioff, Charles W. Dohn, and Donald F. Horton; and October, 1948, by E. Montford Fucik, Charles M. Romanowitz, and Raphael G. Kazmann.

[102] Cons. Engr., Chicago, Ill.

of the effect of bank suction. A ship could approach or pass over such a dike without danger of grounding.

Construction Planning and Methods.—It is quite possible that the occasion never again may arise requiring dredges to dig blasted rock at depths of 145 ft below the water surface. Certainly it has never arisen heretofore. Unusual conditions require and inspire the conception of unusual equipment.

The dipper dredge described evolved naturally from dipper dredges digging from 50 ft to 60 ft deep. The increase to 85 ft is an over-all increase in size. The 500,000-lb bail pull indicates a horizontal thrust of the dipper teeth of from 250,000 lb to 300,000 lb.

The hydraulic dredge has the unusual feature of a pump located well down the ladder. This presents no mechanical difficulties and the digging reaction in a hydraulic dredge is not severe.

A bucket-ladder dredge for digging to a depth of 145 ft is a monstrosity with a 230-ft-long ladder, consisting of two trusses 20 ft deep and many heavy buckets. Bucket-ladder dredges have operated in Europe for decades, digging soft and medium hard material in harbors and rivers to a depth of 40 ft or more. Also, they have been used most successfully in digging for gold, platinum, and tin, sometimes in compacted gravel, but usually in fairly loose material.

Most placer deposits consist largely of coarse sand, gravel, and small boulders, all of which easily enter the buckets of the bucket-ladder dredge and are carried up to the dumping point without loss of water or mineral through leakage. The high central discharge point adapts itself to feeding screens and tables, with stackers located at the stern. Sometimes these dredges excavate bedrock when it contains fissures apt to hold minerals, but usually the bedrock is shale or slate and only about a foot would be excavated. There is no other type of dredge at all suitable for placer work.

The bucket-ladder dredge purchased by the Panama Canal Authorities about 1912 in Scotland utterly failed to dig blasted rock in the Culebra Cut and was replaced as soon as possible by dipper dredges, which were ideal for the work because of the absence of any space or size restriction on the pieces of rock to be handled. Photographs show these dipper dredges raising to the water surface great pieces of rock too large to pass through the dipper. These were lowered into the water, blasted, and the smaller pieces picked up by the dipper and loaded on barges.

With any system of drilling and blasting there are oversize pieces of rock, just as in the case of a primary crusher, where 15% of the rock will be larger than the crusher opening size. If drilling and blasting are adapted to produce rock that can be pushed into a 2-cu-yd bucket and carried up the ladder, what will happen to the inevitable oversize pieces? They cannot be handled by the 2-cu-yd buckets; they cannot be blasted again at that depth. The only solution would be to dig holes and bury them, or to use a grapple for each individual rock.

To equal the digging thrust of the dipper dredge, the spuds of the bucket-ladder dredge would have to provide a reaction of from 250,000 lb to 300,000 lb. As the width of the 2-cu-yd bucket is probably one third of the width of the dipper on the dipper dredge, the thrust could be reduced, but the efficiency is also reduced in the handling of large rock pieces.

The point of discharge on the proposed bucket-ladder dredge is where the buckets turn over the upper tumbler. This point is 50 ft above the water surface and the rock will have to be loaded into the barges by gravity or pan conveyors. In addition, the buckets must be built with a high factor of safety because one broken bucket will often mean the loss of the whole chain, which causes days of delay. About half of the buckets are subjected to the digging strains at all times.

Two other types of dredge should not be overlooked as possible solutions to the problem—the clamshell and the dragline.

Clamshell Dredge.—A clamshell dredge was used at Pearl Harbor, Hawaii, for excavating blasted rock in a seaway where it was impossible to use a dipper dredge because of the large swell. The closing power of the bucket is dependent on its weight, which is necessarily limited, but this type of equipment can excavate with nothing but wire ropes between the dredge and the sea floor.

Dragline Dredge.—Since the publication of this Symposium, there has been devised a dragline dredge which will dig at 90 ft or 148 ft without change, and which will have a 30-cu-yd bucket 12 ft wide and a digging thrust of 250,000 lb by direct pull of the draglines in an almost horizontal direction. The theoretical capacity is less than that of the bucket-ladder dredge, but the actual capacity would probably not be much less. The 30-cu-yd drag bucket would be raised to 10 ft above the water and dumped into the barge under perfect control.

All rock resulting from suitable blasting would be handled by this dredge; the drilling and the blasting would be 40% of those required by the bucket-ladder dredge. The excavation of blasted rock can be accomplished only by the use of a large bucket with teeth and of a large dragline pull at an angle as nearly horizontal as possible at first, and of not more than 45° after opening up a hole or a pit. The dragline dredge would dig as effectively at the 148-ft depth as would the dipper dredge at the 85-ft depth.

Although it may have been considered, no mention is made of the advisability of excavating with the special dredges to a depth of about 130 ft (El. −42) and of leaving the rest of the excavation to El. −60 to be done by regular dredges after the canal is opened for traffic. The difficulty would probably be the disposal of the spoil with the water surface at sea level and with large spoil areas eliminated, but it would advance the date of opening the sea-level canal to traffic.

William Allan,[103] M. ASCE.—In solving the tidal current problem for a sea-level canal at Panama, Messrs. Meyers and Schultz have made a notable contribution to the science and art of hydraulic computation and experimentation.

The results of extended calculations and of model tests confirm the evaluation of marginal limiting velocities computed by the steady-flow method and suggest the desirability of an explanation of the argument of approach and of the procedures followed in this method. The writer cooperated with Boris A. Bakhmeteff, Hon. M. ASCE, in making the preliminary calculations mentioned in the paper (under the heading, "Analyses of Tidal Currents for 1947 Investiga-

[103] Prof. of Civ. Eng. and Dean, School of Technology, The City College of New York, New York, N. Y.

tion") and presents the method used as applied to a conventionalized Panama Canal based on the adopted cross section.

Because the Pacific and Atlantic levels, by which flow in a sea-level canal would be actuated, change continuously with time, the flow is variable (unsteady). The governing equations of motion and of continuity, respectively, are

$$S = \frac{1}{g}\frac{\partial v}{\partial t} + \frac{\partial}{\partial x}\frac{v^2}{2g} + \frac{Q^2}{A^2 C^2 R} \quad \ldots\ldots (4)$$

and

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \quad \ldots\ldots (5)$$

Eq. 4 is the equation of instantaneous dynamic equilibrium, and Eq. 5 is the equation of storage.

A solution using these equations is possible only by adopting a simplifying assumption to make friction vary as the first power of the velocity in selected reaches and periods of time[104,18] or by methods of successive approximations.[8] By these methods a complete solution of a given problem may require many thousand man-hours of computation.[105] The method presented in this discussion, although mentioned in the paper, has not heretofore been published. In the present instance a complete solution is essential, but it is especially desirable to know the limiting maximum tidal currents. These limiting values and other features may be found in from 25 to 30 man-hours by adopting justifiable simplifications which enable one to make a solution based upon varied (steady) flow. This assumes no change of flow characteristics with time, thereby making the local acceleration slope, $\frac{1}{g}\frac{\partial v}{\partial t}$, zero and removing storage from consideration. These conditions are satisfied approximately when the Pacific tide is at about its highest and its lowest level, respectively, at which points in the cycle the change of elevation with time is small compared to the total rise or fall of the tide. The interval of time to be used in observing the comparative smallness of change of elevation is the period required for a change (cause) at the Pacific end to be felt (effect) at the Atlantic end. It is the time required for a translation wave to travel from the Pacific to the Atlantic. This impulse or wave travels at a rate of $(c + v)$ with respect to the bank when flow is from the Pacific end toward the Atlantic end and at the rate of $(c - v)$ when flow is from the Atlantic toward the Pacific. v being the flow velocity and c, the celerity of propagation of a wave in still water. This propagation rate is evaluated by

$$c = \sqrt{g\frac{A}{b}} \quad \ldots\ldots (6)$$

in which b is the width of the canal at the surface level and A/b is the average depth. The effect of friction on the wave travel, acknowledged to be very small, is omitted.

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[104] "Leçons sur la Théorie des Marées," by Maurice Lévy, Gauthiers-Villars, Paris, 1898.

[18] "The Cape Cod Canal," by William Barclay Parsons, *Transactions*, ASCE, Vol. LXXXII, 1918, p. 138.

[8] "Tidal Hydraulics," by George B. Pillsbury, *Professional Paper No. 34*, Corps of Engrs., U. S. Govt. Printing Office, Washington, D. C., 1940.

[105] "Analogy Between Tides and A. C. Electricity," by Johan Van Veen, *The Engineer*, November 28, 1947, p. 498.

The conventionalized canal has a channel 220,000 ft long; a bottom width of 560 ft; side slopes of 1 on 1; a depth at the Pacific end of 60 ft at low tide and 80 ft at high tide; a constant depth at the Atlantic end of 60 ft (no tide); a bottom slope downward from the Atlantic to the Pacific of 10 ft in 220,000 ft ($= 0.455 \times 10^{-4}$); and Manning's $n = 0.024$. After completing the basic computations, the ± 1-ft tidal range at the Atlantic end used in the Panama studies was considered, local effects at each end were assessed, and adjustments were made.

In steady flow, when the Pacific stage is below that of the Atlantic, flow is toward the Pacific and in the direction of the bottom (sustaining) slope. Discharges (delivery curves) and surface profiles were computed in this case by use of tables of varied flow functions.[106] When the Pacific stage is above that of the Atlantic, flow is toward the Atlantic and against an adverse bottom slope.[107] For this case no tables were available, so the integral functions were calculated by step-by-step integration and then used to compute the discharges and surface profiles. Steady flow toward the Pacific, if the stage there were held at −10 ft, would be 214,000 cu ft per sec at a uniform depth of 60 ft; and the average velocity of flow would be $v = Q/A = 214{,}000/37{,}200 = 5.75$ ft per sec. At this limit $c = \sqrt{32.2 \times 37{,}200/680} = 42$ ft per sec. Hence, at low Pacific tide, changes there will be transmitted toward the Atlantic at the rate of about $42 - 5 = 37$ ft per sec, and the "time lag" required for a change originating at the Pacific to produce an appropriate effect at the Atlantic will be about $t_w = L/(c - v) = 220{,}000/37 = 5{,}950$ solar sec = 1.65 solar hr = 1.60 lunar hr.

Steady flow toward the Atlantic, if the Pacific stage were held at + 10 ft, would be 267,000 cu ft per sec and the surface profile would be of the M_2 type.[106] An approximate over-all flow velocity on the basis of a 70-ft average depth would be $v = 267{,}000/44{,}100 = 6.05$ ft per sec and for this same average depth $c = \sqrt{32.2 \times 44{,}100/700} = 45$ ft per sec. Hence, changes in Pacific stage at high levels will be transmitted toward the Atlantic at the rate of about $45 + 6 = 51$ ft per sec and will be felt there approximately $t_w = L/(c + v) = 220{,}000/51 = 4{,}320$ solar sec = 1.20 solar hr = 1.16 lunar hr later. Referring to Fig. 26 it will be noted that in 1.60 lunar hr at Pacific low tide the stage falls from − 9.14 ft to − 10 ft and then rises again to − 9.14 ft, a vertical change of 0.86 ft. In one half of this time lag the change down to the lowest level is only 0.22 ft. At Pacific high tide, in 1.16 lunar hr the stage rises from + 9.54 ft to + 10 ft and then falls again to + 9.54 ft, a change in elevation of 0.46 ft. In 0.58 lunar hr at the peak the change is only 0.12 ft.

For comparison, a calculation of steady flow toward the Atlantic from Pacific stage +9.54 ft gives 258,000 cu ft per sec as against 267,000 cu ft per sec from stage +10 ft; toward the Pacific with a −9.14-ft stage a similar calculation gives 207,000 cu ft per sec as against 214,000 cu ft per sec with −10-ft stage. These values and the corresponding surface profiles permit an assessment of the total effect of the local accelerations, which were neglected. At the upper and also at the lower limits of the Pacific tide this value is about 5%

[106] "Hydraulics of Open Channels," by Boris A. Bakhmeteff, McGraw-Hill Book Co., Inc., New York, N. Y., 1932.

[107] "Varied Flow in Open Channels of Adverse Slope," by Arthur E. Matzke, *Transactions*, ASCE, Vol. 102, 1937, p. 651.

of the total fall over the entire length of the canal. In terms of discharge less than 3% error is indicated.

Several steady-flow surface profiles and discharges are shown in Fig. 112. These suggest a method of tracing approximate successive instantaneous surface profiles near the upper and lower limits of the Pacific tide. For example, if the Pacific level were imagined to stand for a long time at El. 79.54 (marked, "1/2 wave travel time") the corresponding steady-flow surface profile would be formed. If the Pacific level were now free to follow the tide curve it would rise in 0.29

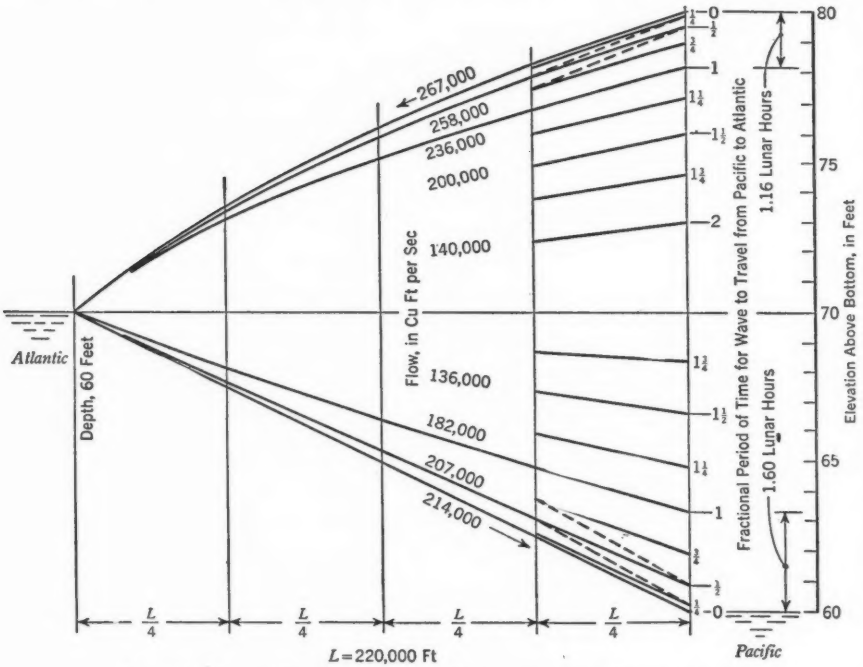

FIG. 112.—STEADY-FLOW CURVES FOR CONVENTIONALIZED CANAL SECTION
(Bottom Slope = 0.455×10^{-4}, Manning's $n = 0.024$,
Bottom Width = 560 Ft, and Side Slope, 1 on 1)

lunar hr (the time for a wave to travel one fourth the length of the canal) to the point marked "1/4" (El. 79.88) and in that time interval the effect of the rise would extend to a point $L/4$ along the canal. The surface on the original "1/2" curve at that $L/4$ point would begin to rise only at that instant. Consequently, the approximate water surface would extend from the "1/4" curve at the Pacific to the "1/2" curve at a point L_4 in from the Pacific and the local influx would correspond to this surface slope. In the next interval of 0.29 lunar hr the Pacific level would reach the "0" curve at El. 80, the surface would reach the "1/4" curve at the $L/4$ point, and the effect of the Pacific rise begun 0.58 lunar hr previously would now have extended to the midlength of the canal on the "1/2" curve, and so on. Only after a total interval of 1.16 lunar hr from the start will the effect of the initial movement at the Pacific end of the "1/2" curve be felt at the Atlantic end. The efflux at the Atlantic will correspond to the instan-

taneous slope of the water surface there and will reflect the effect of the change in Pacific level initiated 1.16 lunar hr previously. This time lag is borne out by the model tests. By this procedure, under the assumption of zero tidal range there, the limiting water-surface slopes at the Atlantic are those corresponding to the extreme upper and lower steady-flow profiles with their respective flows of 267,000 cu ft per sec out of the canal, and 214,000 cu ft per sec into the canal. Adjustment of these profiles to account for the tidal range of ± 1 ft used in the tests increases the computed flows to 277,000 cu ft per sec and 226,000 cu ft per sec, respectively, with velocities of 4.4 knots out of the canal and 3.6 knots into the canal at the Atlantic end compared to 4.4 knots and 3.7 knots by test on the canal model.

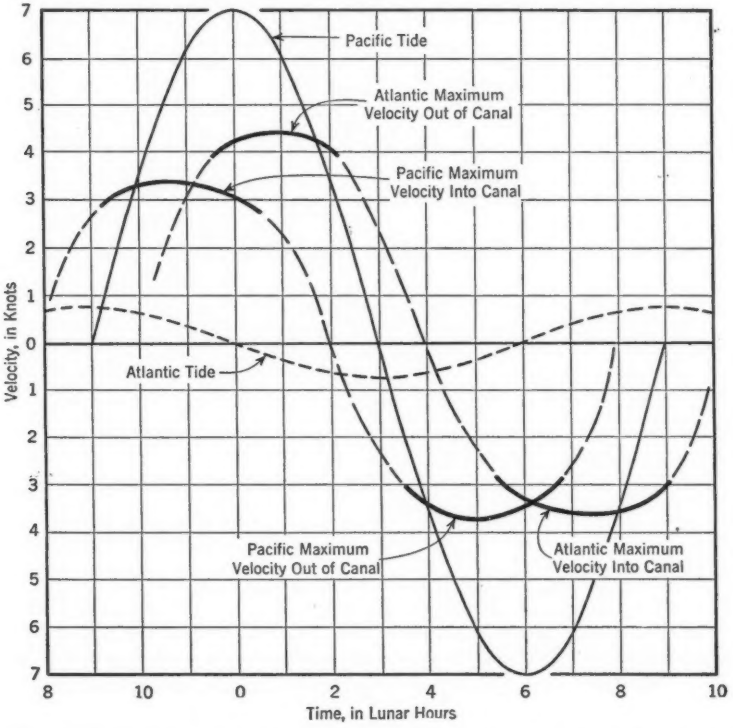
Pacific Tide
Atlantic Maximum Velocity Out of Canal
Pacific Maximum Velocity Into Canal
Atlantic Tide
Pacific Maximum Velocity Out of Canal
Atlantic Maximum Velocity Into Canal
Velocity, in Knots
Time, in Lunar Hours
7 6 5 4 3 2 1 0 1 2 3 4 5 6 7
8 10 0 2 4 6 8 10

FIG. 113.—CURRENT VELOCITIES AT ATLANTIC AND PACIFIC ENDS OF CONVENTIONALIZED CANAL

A succession of approximate instantaneous local water-surface slopes at the Pacific end appropriate to rising and falling tides near high water and low water gives corresponding maximum flows of approximately 284,000 cu ft per sec into the canal near high tide, and 243,000 cu ft per sec out of the canal near low tide. These values include a small adjustment to account for the ± 1-ft tide range at the Atlantic and give maximum velocities of 3.4 knots into the canal and 3.7 knots out of the canal at the Pacific end, as compared to 3.3 knots and 3.7 knots, respectively, by model test. The order of some of the approximate instantaneous water surfaces is shown by dashed lines in Fig. 112.

Before adjusting for the Atlantic tide range, a check was made between Pacific influx and Atlantic efflux and the storage between approximate instantaneous water-surface profiles for the period from 0.58 lunar hr before high tide to 0.58 lunar hr after high tide at the Pacific end. The result was excellent.

Velocity values determined by this calculation for stations 0 (Atlantic) and 2,200 (Pacific) of the conventionalized canal are shown in Fig. 113, in which the tide cycle is shown to emphasize the time relationships.

BORIS A. BAKHMETEFF,[108] HON. M. ASCE.—The investigations reported on in the paper by Messrs. Meyers and Schultz set a landmark in the studies of tidal currents in ship canals. In each major engineering project there are certain crucial questions to which clear and dependable answers are required at an early stage in order to judge the feasibility of the whole enterprise. In the instance of the sea-level canal over the Panama Isthmus, in addition to the problem of stable slopes and comprehensive methods of mass excavation, the question was to know the size and the regimen of the currents to be engendered by the tides. The problem is one of unsteady motion, variable in time. The general differential equations are easily written and have been known for years. Thus far, however, satisfactory mathematical solutions in the case of variable flow only could be made either by disregarding the effects of friction (as in the theory of surges and translational waves), or by neglecting inertial effects (as in "hydraulics of floods"). In tidal canals such as the Panama Canal, both inertia and friction exercise comparable influence and a reliable solution requires accounting for the effects of both.

There are no direct mathematical methods that permit a complete and exact integration of the equations under such circumstances. Accordingly, in the past, each time a major ship canal was envisaged outstanding mathematical talent was called upon to offer solutions.[19,109] At best, however, simplified assumptions were necessitated as substitutions for the actual laws of nature. Another method of attack is by gradual arithmetical integration carried out by successive approximation.[8] More recently, recourse has been suggested to the electric analogy.[105]

Notwithstanding the remarkable ingenuity of these methods, there is a practical drawback in that it has not been possible to appraise beforehand the magnitude and even the sense of the possible error. In other words, it was not known whether the values arrived at were greater or less than those to be anticipated in practice. On the other hand, even in the preliminary stages of the canal design it is imperative to know with certitude at least the extreme limits within which tidal currents are bound to occur. The writer vividly recalls the sense of oppressive responsibility when faced with the first stages of the work early in 1946, the final results of which are reported on now with such confident certainty by Mr. Meyers. Indeed, it was for the very limited purpose of obtaining marginal current values, and with no claim for a complete solution of the tidal regimen, that the writer conceived the approach referred to in the paper, which amounts to checking the flow at the extreme tide

[108] Prof., Civ. Eng., Columbia Univ., New York, N. Y.

[19] *Comptes Rendus*, French Academy of Sciences, Vol. 104, May 31, 1887, p. 1484.

[109] "The Cape Cod Canal," by William Barclay Parsons, *Transactions*, Vol. LXXXII, 1918, p. 105.

levels by methods of steady varied motion. It was self-evident, of course, that the outflow in variable motion at the Cristobal end of the canal, where conditions are crucial, would under all circumstances be below the maximum figures computed for steady conditions. With that, however, the possible excess of the calculated values over actual flow values could be appraised only approximately.

A complete and exhaustive answer to all the questions concerning tidal currents could be confidently given solely by experimenting on a suitable model, and the decision to take recourse to model investigations in the preliminary stage of design stands as an eloquent testimony to the vision and courage of the Supervising Engineer under the egis of the Governor of The Panama Canal. The fact is that in order to give effective and indisputable answers, the model had to be undistorted and, moreover, built on a sufficiently large scale. Mr. Meyers and his assistants deserve all possible credit for the design, construction, and operation of the model, which is of unprecedented type and scale.

With the results described in the paper at hand, it may be stated confidently that the sea-level canal project, as submitted by the Governor to Congress, has answered the problems concerning tidal flow in a complete and indisputable fashion. Actually, it is the first time in the history of engineering that such an answer has been given with certitude and finality.

The work reported on by Messrs. Meyers and Schultz carries further significance in that, for the first time, observational data are available to compare and check results of computation by the different approximate methods. Thus, as stated in the paper (under the heading, "Currents in an Open Sea-Level Canal") the method devised by George B. Pillsbury, M. ASCE, after the necessary successive approximations, yields an over-all account of the regimen in close accord with actual facts. The writer was most gratified to find, also, that the marginal figures of maximum flow obtained by his own approach only negligibly exceeded the observed velocities. On the whole, engineering science has moved a step forward in its knowledge of tidal phenomena and in the appraisal of methods to calculate and predict the course of tidal phenomena.

A serious point of weakness, which Mr. Meyers was compelled to face, is the lamentable lack of observational data relating to frictional resistance in large canals. The values in Table 19, assembled with a great deal of effort, clearly exhibit how negligent the profession has been in making the necessary observations on existing canals of large size—one more testimony to the urgency of the task assigned by the Society to the Committee on Hydraulic Data and Facts of its Hydraulics Division.

It is only natural that under the pressure of time the Panama Canal model could be used only for obtaining answers of immediate practical character. As built, however, the model offers unprecedented facilities for systematic study of variable channel flow subject to frictional and inertial effects. Knowledge in this field is very meager and quite important. Naturally, for carrying out such work the model would require more delicate instrumentation than has been available. However, the basic equipment is there and it is confidently hoped that the model will be preserved and put to use to continue its valuable pioneer work.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

HIGHWAY BRIDGE FLOORS
A SYMPOSIUM

Discussion

BY RETO FURRER

RETO FURRER.[29]—Part B of the paper by Professor Siess shows some strain diagrams (Figs. 25 and 26) obtained from load tests on composite beams. Using a novel method of analysis,[30] these test results have been evaluated in the Department of Civil Engineering at the Swiss Federal Institute of Technology in Zurich, Switzerland. It was possible, for example, to take into account the deformations of the shear connectors in computing the stresses for the composite section, differing in this respect from the classical theory of bending.

Fig. 39 is symbolic of a composite beam, the upper concrete part (designated by subscripts *c*) being separated from the lower steel part (designated by subscripts *s*), for analytical purposes.

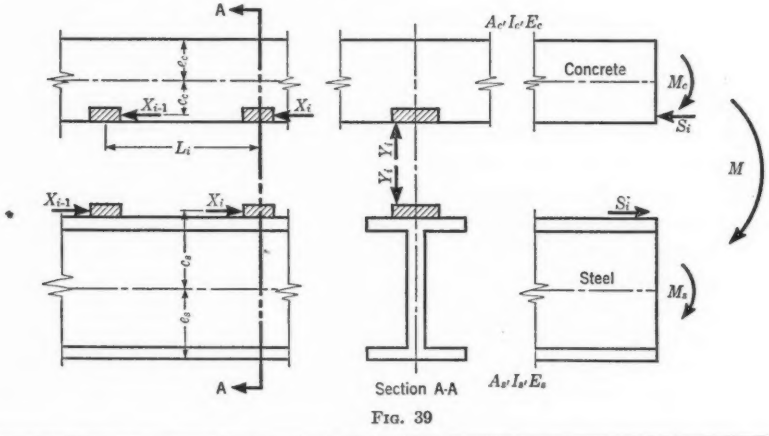

FIG. 39

NOTE.—This Symposium was published in March, 1948, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: September, 1948, by Searcy B. Slack; and October, 1948, by E. W Wendell, and H. Tachau.

[29] Candidate in Civ. Eng., Swiss Federal Inst. of Tech., Zurich, Switzerland.

[30] "Composed Beams," by F. Stüssi, *Publications*, International Assn. for Bridge and Structural Eng. Zurich, Switzerland, Vol. 8, 1947, p. 249.

The unknown shearing forces at a section A–A resolved into the horizontal component X_i and the vertical component Y_i. The symbol S_i represents the sum of all forces X_i from the left support to section A–A. The vertical components Y_i are expressed by their moments M_{yi} about the centroidal axis of both beams. In the simple system with $X = 0$ and $Y = 0$, the external moment M is distributed to the two beams as follows (assuming that the deflections of the two parts are equal):

$$M_{0c} = M \frac{n_c I_c}{I_{n0}} \quad \text{and} \quad M_{0s} = M \frac{n_s I_s}{I_{n0}} \ldots\ldots\ldots\ldots (16)$$

$$n_c = \frac{E_c}{E} \quad \text{and} \quad n_s = \frac{E_s}{E} \ldots\ldots\ldots\ldots\ldots (17a)$$

in which E is an arbitrary value of the modulus of elasticity and

$$I_{n0} = n_c I_c + n_s I_s \ldots\ldots\ldots\ldots\ldots\ldots (17b)$$

The moments acting at the center of gravity of the two beams are

$$M_c = M_{0c} + M_y + S_i c_c \ldots\ldots\ldots\ldots\ldots (18a)$$

and

$$M_s = M_{0s} - M_y + S_i c_s \ldots\ldots\ldots\ldots\ldots (18b)$$

In Fig. 40, let f_{cg} and f_{sg} be, respectively, the stresses at the centroids of the upper beam and of the lower beam; $f_{cg} + \Delta f_c$ and $f_{sg} + \Delta f_s$ be the stresses at the upper edge of the upper beam and at the lower edge of the lower beam; and e_c and e_s be the distances from the upper centroid to the upper edge and from the lower centroid to the lower edge.

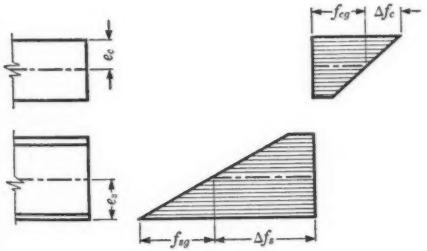

FIG. 40

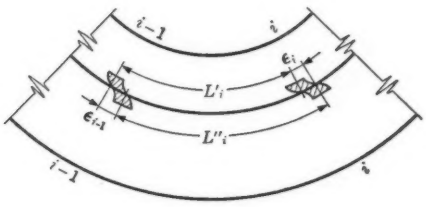

FIG. 41

Fig. 41 shows the displacements resulting from the deflection of the composite beam. Let L_i be the spacing of shear connectors before deflection; ϵ_{i-1} and ϵ_i represent the deformation of shear connectors; and

$$L'_i = L_i + \Delta L'_i \ldots\ldots\ldots\ldots\ldots\ldots (19a)$$

and

$$L''_i = L_i + \Delta L''_i \ldots\ldots\ldots\ldots\ldots\ldots (19b)$$

are the changed spacings, respectively, at the lower edge of the upper beam and at the upper edge of the lower beam. From the relation $\frac{\Delta L}{L} = \epsilon = \frac{f}{E}$

the changes in length are determined with the aid of Fig. 40 as: The bottom fiber of the upper beam—

$$\Delta L'_i = \frac{L_i}{E_c}\left(- f_{cg} + \frac{\Delta f_c}{e_c} c_c \right) \ldots\ldots\ldots\ldots\ldots\ldots (20a)$$

and the top fiber of the lower beam—

$$\Delta L''_i = \frac{L_i}{E_s}\left(f_{sg} - \frac{\Delta f_s}{e_s} c_s \right) \ldots\ldots\ldots\ldots\ldots\ldots (20b)$$

The deformation of the shear connector is assumed proportional to the force on the connector, X_i; therefore,

$$\epsilon_{i-1} = \frac{X_{i-1}}{C_{i-1}} = \frac{S_i - S_{i-1}}{C_{i-1}} \ldots\ldots\ldots\ldots\ldots\ldots (21a)$$

and

$$\epsilon_i = \frac{X_i}{C_i} = \frac{S_{i+1} - S_i}{C_i} \ldots\ldots\ldots\ldots\ldots\ldots (21b)$$

in which the quantity C represents the resistance of the shear connector to deformation.

The geometrical conditions of elasticity require that

$$\Delta L'_i + \epsilon_i = \Delta L''_i + \epsilon_{i-1} \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

Substituting Eqs. 20 and 21 in Eq. 22,

$$\frac{L_i}{E_c}\left(- f_{cg} + \frac{\Delta f_c}{e_c} c_c \right) + \frac{S_{i+1} - S_i}{C_i} = \frac{L_i}{E_s}\left(f_{sg} - \frac{\Delta f_s}{e_s} c_s \right) + \frac{S_i - S_{i-1}}{C_{i-1}} \ldots (23)$$

In Eqs. 18 substitute

$$M_{0c} = \frac{\Delta f_c}{e_c} I_c \qquad \text{and} \qquad M_{0s} = \frac{\Delta f_s}{e_s} I_s \ldots\ldots\ldots\ldots\ldots (24)$$

The equation of equilibrium $M = M_c + M_s$ then becomes

$$M = \frac{\Delta f_c}{e_c} I_c + \frac{\Delta f_s}{e_s} I_s + S_i(c_c + c_s) \ldots\ldots\ldots\ldots\ldots (25)$$

In Eq. 25, replace $(c_c + c_s)$ by d, the distance between the two centroidal axes.

The elasticity condition of "equal deflection of both individual beams" may be replaced in the middle of the panel by the condition that the strain distribution lines for each beam have the same slope. According to Fig. 40,

$$\frac{\Delta f_c}{E_c e_c} = \frac{\Delta f_s}{E_s e_s} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

Substituting Eq. 26 in Eq. 25 and simplifying,

$$\frac{\Delta f_c}{E_c e_c} = \frac{M - S d}{E_c I_c + E_s I_s} E_c \ldots\ldots\ldots\ldots\ldots\ldots (27a)$$

and

$$\frac{\Delta f_s}{E_s e_s} = \frac{M - S d}{E_c I_c + E_s I_s} E_s \ldots\ldots\ldots\ldots\ldots\ldots (27b)$$

The total force S can be expressed as

$$S = f_{cg} A_c = f_{sg} A_s \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

Furthermore, the following abbreviations can be introduced:

$$I_{n0} = n_c I_c + n_s I_s \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (29a)$$

$$A_n = 2 \frac{n_c A_c \times n_s A_s}{n_c A_c + n_s A_s} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (29b)$$

and

$$I_n = I_{n0} + A_n \frac{d^2}{2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (29c)$$

Eq. 27 is then substituted in Eq. 23, and the following fundamental equation of the problem is obtained:

$$-\frac{S_{i-1}}{C_{i-1}} + S_i \left(\frac{1}{C_{i-1}} + \frac{1}{C_i} + 2 \frac{L_i I_n}{E I_{n0} A_n} \right) - \frac{S_{i+1}}{C_i} = \frac{L_i d}{E I_{n0}} M_{im} \ldots . (30)$$

in which M_{im} is the average of the external moments over the distance between two shear connectors.

Eq. 30, set up as a system of equations, yields the unknown quantity S_i. The stresses in the middle of the panel can be computed using

$$f_{cg} = \frac{S}{A_c} \quad \text{and} \quad f_{sg} = \frac{S}{A_s} \ldots\ldots\ldots\ldots\ldots\ldots (31)$$

Thus,

$$\Delta f_c = \frac{M - S d}{I_{n0}} n_c e_c \quad \text{and} \quad \Delta f_s = \frac{M - S d}{I_{n0}} n_s e_s \ldots\ldots . (32)$$

By taking into account the moment M_y resulting from the vertical shearing forces, the distribution of stresses over the entire composite beam can be determined.

The strain diagrams in Fig. 25 were evaluated by these theoretical considerations. First, the following values of $n = \frac{E_s}{E_c}$ were determined from the relative values of the strains at the centroidal axis: For beam T1, $n = 9.4$; for beam T2, $n = 14.0$; and, for beam T3, $n = 12.5$. It is evident from these values that the compressive strength of the concrete slab for beam T2 was less than that for beam T1.

To determine the deformation resistance factor, C, for shear connectors, Eq. 30 was solved for various values of C. Assuming that the C-values for beams T1 and T2 must be equal (since the shear connectors were the same) and that the C-value for beam T3 must be smaller, and also based on test data from the Swiss Federal Testing Laboratory, the following C-values (in tons per centimeter) were obtained: For beams T1 and T2, $C = 400$; and, for beam T3, $C = 200$.

The stresses computed by the foregoing procedure are compared with those computed by the Symposium tests and with these computed by the classical theory of bending in Table 6. (The classical theory of bending does not consider the deformation of the shear connectors.)

TABLE 6.—COMPARISON OF STRESSES (KILOGRAMS PER SQUARE CENTIMETER = 14.23 LB PER SQ IN.) IN A COMPOSITE BEAM FOR A LOAD OF 5,200 LB AT MIDSPAN

Method	Section A, 3.75 In. from Midspan				Section B, 19 In. from Midspan			
	Upper Edge		Lower Edge		Upper Edge		Lower Edge	
	Stress	Diff.[a]	Stress	Diff.[a]	Stress	Diff.[a]	Stress	Diff.[a]
Beam T1, Fig. 25 (n = 9.4 and C = 400 Tons per Cm)								
Symposium tests	85.5	....	1,505	....	....	....	....	....
Stüssi method[b]	90.8	+ 6.2	1,489	− 1.1	....	....	....	....
Classical theory of bending	86.3	+ 0.9	1,421	− 5.6	....	....	....	....
Beam T2, Fig. 25 (n = 14.0 and C = 400 Tons per Cm)								
Symposium tests	82.6	....	1,690	....	32.2	....	627	....
Stüssi method[b]	78.0	− 5.6	1,606	− 5.0	30.5	−5.3	633	+ 1.0
Classical theory of bending	74.7	− 9.6	1,486	−12.1	29.1	−9.6	573	− 8.6
Beam T3, Fig. 25 (n = 12.5 and C = 200 Tons per Cm)								
Symposium tests	96.0	....	1,640	....	29.4	....	628	....
Stüssi method[b]	81.8	−14.8	1,596	− 2.7	31.4	+6.8	611	− 2.7
Classical theory of bending	73.0	−24.0	1,375	−16.2	30.4	+3.4	563	−10.4

[a] Percentage difference from values observed in the Symposium tests. [b] "Composed Beams," by F. Stüssi, *Publications*, International Assn. for Bridge and Structural Eng., Zurich, Switzerland, Vol. 8, 1947, p. 249.

It may be noted that the Stüssi method of analysis results in a stress distribution which approximates very closely the actual stress distribution obtained in the tests reported by the Symposium authors. Therefore, it offers a good basis for the further evaluation of tests and presents the civil engineer with the possibility of an improved method of calculation.

Acknowledgment.—The basic method of evaluating Professor Siess' laboratory results was developed by F. Stüssi, professor of steel construction at the Swiss Federal Institute of Technology.[34]

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

STABILITY OF THIN-WALLED COMPRESSION MEMBERS

Discussion

BY BORIS BRESLER, AND GEORGE WINTER

BORIS BRESLER,[10] JUN. ASCE.—The emphasis in this paper is placed, quite properly, on the determination of the ultimate strength of thin-walled compression members. Much progress has been made in the design of light gage steel structures for ordinary service loads. The publication of specifications[11] in 1946 brought together much valuable information and established standard procedures for stress analysis. These specifications, however, provide for the computation of the maximum allowable safe load, and not the ultimate strength of the member, which may be desired under special conditions. Therefore, Mr. Wimer's contribution is particularly welcome. The method proposed by the author has much appeal for the designer because of its directness and simplicity. Unfortunately, some assumptions and limitations of the proposed method restrict its application to special cases.

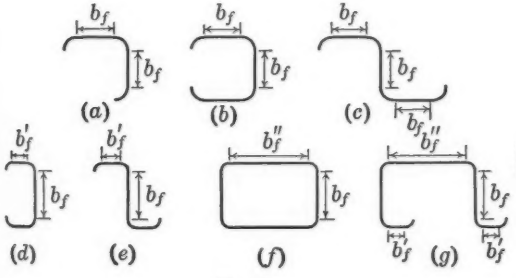

FIG. 9

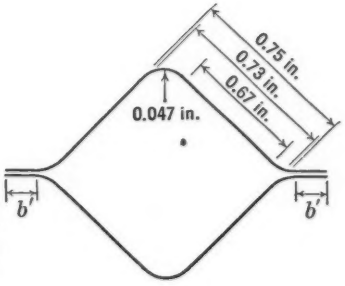

FIG. 10

All the tests reported by Mr. Wimer were made on square tubes, so that the flat widths of all elements b_f (see Fig. 4) are the same. Actually, the shapes used in practice may be quite different from the square shape. Some typical

NOTE.—This paper by H. Dudley Wimer, Jr., was published in May, 1948, *Proceedings*.

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[11] "Specification for the Design of Light Gage Steel Structural Members," A.I.S.I., New York, N. Y., April, 1946.

sections are shown in Fig. 9. The validity of the relationship between σ_a and b_f/h obtained on the basis of tests on specimens of only one shape is open to question. In fact, the relationship has no real significance for sections such as shown in Figs. 9(*d*) to 9(*g*), because the value of b_f/h depends on the particular value of b_f used in the expression.

Assuming that relationship $\sigma_a\ (b_f/h) = 16$ is valid for all sections with equal flat width elements, the author's method can be simplified as follows:

$$P_t = \sigma_a\,(\text{UTS})\,A_g \qquad (11a)$$

and

$$P_t = \sigma_c\,(\text{UTS})\,A_e \qquad (11b)$$

in which UTS is the ultimate tensile strength; A_g is the gross cross-sectional area of the member exclusive of welding flanges; A_e is the effective area of the member; σ_a is determined from Fig. 5 or computed as $16\ h/b_f$; and σ_c is determined from Fig. 3 for a given $\frac{D}{h}$-ratio. From Eqs. 11,

$$A_e = \frac{\sigma_a}{\sigma_c} A_g \qquad (12)$$

and P_t can be computed from either Eq. 11*a* or Eq. 11*b*.

Consider the example given by the author (Fig. 10). Neglecting the welding flanges b', the gross area A_g is (with very small error) equal to: $A_g = 4 \times 0.073 \times 0.010 = 0.0292$ sq in.; $\sigma_a = \frac{16}{b_f/h} = \frac{16}{0.67/0.010} = 0.249$; and $\sigma_c = 1.0$ (from Fig. 3). Hence, from Eq. 12, $A_e = 0.249 \times 0.0292 = 0.00727$ sq in. which is precisely the area that would have been obtained by the author if welding flanges had been neglected.

It is thus seen that the computation of effective width $2\,b_e$ of each flat is unnecessary if the section has no welding flanges or other straight stiffening elements free along one edge. Thus, the author's procedure can be simplified in many cases.

To evaluate the strength of the welding flanges b' (Fig. 10) the author arbitrarily assumes the effective width of each edge as b_e—that is, half as effective as a flat element supported along both edges. For all general cases, this assumption is unconservative, since the strength of an element which is free along one edge and supported along the other edge, is less than one half the strength of an element supported along both edges. Values given by the author for the example under consideration are in agreement with this statement: P_{test} is 1,690 lb, against 1,715 lb computed as failing load. Even though little error is introduced in the particular cross section discussed by the author, the general use of this assumption is not justified.

It must also be stated that Eq. 7 as derived by the author is not correct for two reasons. Firstly, the assumption made in Eq. 2 that $\sigma_a = \frac{16}{b/h}$ is not justified because b_f is not equal to b, and the test data plotted in Fig. 5 indicate

that $\sigma_a = \dfrac{16}{b_f/h}$. Secondly, the assumption preceding Eq. 4 that $S_a\, b\, h = 0.25\, P_t$ is true only for square tubes. Thus Eq. 7, even though corrected for the discrepancy between b_f and b, would hold only for square tubes and would not solve, as the author claims (following Eq. 8*b*): "The general problem of predicting the failing load for the case of local instability of stainless-steel compression members * * *."

The author's experimental data can be used, however, to determine a general expression for the effective width $2\, b_e$, assuming that the flat elements of the section are all equal. Consider a cross section consisting of n flats and m bends (for example, a channel shown in Fig. 9(*b*) would have three flats and four bends). Then, if A_b is the area of one bend,

$$P_t = S_a\,(n\, b_f\, h + m\, A_b) = \sigma_a\,(\text{UTS})\,(n\, b_f\, h + m\, A_b) \ldots\ldots (13a)$$

Also,

$$P_t = S_c\,(n\, 2\, b_e\, h + m\, A_b) = \sigma_c\,(\text{UTS})\,(n\, 2\, b_e\, h + m\, A_b) \ldots\ldots (13b)$$

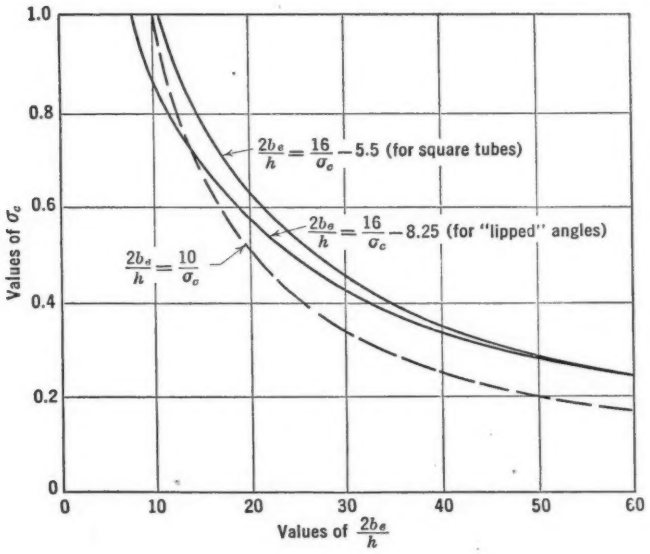

FIG. 11

Solving Eqs. 13 for $2\, b_e$, and substituting $16\, h/b_f$ for σ_a,

$$2\, b_e = \frac{16}{\sigma_c} + \frac{m\, A_b}{n}\left(\frac{1}{b_f} - \frac{1}{h}\right) \ldots\ldots\ldots\ldots\ldots (14a)$$

Assuming $A_b = 5.5\, h^2$, which is based on a radius of bend equal to $3\, h$,

$$2\, b_e = \frac{16}{\sigma_c} + \frac{5.5\, m}{n}\left(\frac{h^2}{b_f} - h\right) \ldots\ldots\ldots\ldots\ldots (14b)$$

Since h^2/b_f is negligible as compared to h,

$$2\,b_e = \frac{16}{\sigma_c} - \frac{5.5\,m\,h}{n} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14c)$$

Eq. 14*c* is plotted for two sections in Fig. 11, and these curves are compared with the curve of Eq. 7 derived by the author. From this comparison it is seen that the use of Eq. 7 is not conservative for some cases.

To summarize this discussion:

1. The method presented by the author is limited to shapes with flat elements of equal width. Any extrapolation of this method to other cross sections must be substantiated by additional tests.
2. In many cases, when the method is applicable, the procedure can be simplified. The effective area A_e can be computed directly from Eq. 12 without computing the effective width $2\,b_e$ of each flat.
3. The assumption that a flat element supported only along one edge is generally half as effective as an element supported on both edges is not conservative.
4. If the effective width $2\,b_e$ is desired, the method presented by the author must be modified to correct for a discrepancy between b_f and b in Eq. 2 and also for the shape of the cross section.

GEORGE WINTER,[12] M. ASCE.—The use of an effective design width for thin-walled compression members, as discussed in this paper, has gained increasing acceptance in standard structural design specifications in recent years. The February, 1946, specifications[13] of the American Institute of Steel Construction prescribe the use of an effective width for compression plates whose width exceeds 40 *t*. The April, 1946, specification[11] of the American Iron and Steel Institute (A.I.S.I.) presents a similar approach in a much more detailed and specific manner, in view of the decisive role of local buckling in the performance of light gage steel members. Both of these specifications apply essentially to mild structural steels, whereas Mr. Wimer's informative tests refer to a hard, high alloy steel.

The effective width stipulations of the second of the previously mentioned specifications[11] are based on the writer's experimental work in this field.[14] It is of practical interest to investigate the degree of agreement or discrepancy between Mr. Wimer's results for hard alloy steels and the writer's results for mild carbon steels.

The writer's findings, in a later, slightly modified form,[15,16] were expressed as follows: Thin, flat compression elements are fully effective ($B_e = b$) if their

[12] Prof. and Head, Dept. of Structural Eng., Cornell Univ., Ithaca, N. Y.

[13] "Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings," A.I.S.C., New York, N. Y., February, 1946.

[14] "Strength of Thin Steel Compression Flanges," by George Winter, *Transactions*, ASCE, Vol. 112, 1947, p. 527.

[15] "Strength of Thin Steel Compression Flanges," by George Winter, *Reprint No. 32*, Cornell Univ. Eng. Experiment Station, Ithaca, N. Y., October, 1947, p. 53, Appendix.

[16] "Performance of Thin Steel Compression Flanges," by George Winter, *Preliminary Publication*, 3d Cong., International Assn. for Bridge and Structural Eng., Liége, Belgium, 1948.

width to thickness ratio does not exceed

$$\frac{b}{h} = 0.95\sqrt{\frac{E}{s}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

For elements with b/h-ratios exceeding that given by Eq. 15, the effective width B_e can be computed with good accuracy from

$$B_e = 1.9\,h\sqrt{\frac{E}{s}}\left(1 - 0.475\,\frac{h}{b}\sqrt{\frac{E}{s}}\right) \quad \ldots\ldots\ldots\ldots\ldots (16)$$

in which s, as in the author's notation, is the edge stress; E is the modulus of elasticity; and B_e is the total effective width (that is, $B_e = 2\,b_e$ in the author's notation).

To apply Eqs. 15 and 16 to 18–8 stainless steel it is necessary to refer to the stress-strain curve of that material. The writer was able to obtain this information only for the very similar 18–7 stainless steel. For 0.02 in. thick, 18.3% cold reduced sheet steel, the properties[17] are practically identical with those given by the author for the 18–8 stainless steel (tensile strength = 186.3 kips per sq in., and E = 26,600,000 lb per sq in.). The slope of the stress-strain curve at high stresses differs materially from this value, resulting in a tangent modulus of about 11,000,000 lb per sq in. at a stress of 110 kips per sq in., and one of about 8,000,000 lb per sq in. at a stress of 120 kips per sq in. Definite data for still higher stresses are not available to the writer; but, from the general shape of the curves of similar steels, it can be assumed that a value of about 7,000,000 lb per sq in. should represent a reasonable approximation up to stresses not too close to the ultimate.

On this basis a comparison can be made between Mr. Wimer's results and those of the writer. The author's tests apparently cover a width to thickness range from about 19 to 125. By substitution of these values, and of the appropriate magnitude of E for various stresses, Eq. 16 determines magnitudes of B_e which can be compared directly with Mr. Wimer's results. This comparison is shown in Fig. 12, in which separate curves are drawn for b/h = 19 and b/h = 125, computed, as indicated, from Eq. 16. The curve for b/h = 19 is discontinued at a stress of 66.5 kips per sq in., because, according to Eq. 15, elements with b/h = 19 are fully effective ($B_e = b$) at stresses below that value.

The findings depicted in Fig. 12 can be stated as follows: In contrast to Mr. Wimer's formula, Eq. 16 indicates that the effective width depends not only on the stress, but also on the flat width ratio b/h, although this influence is small at the high stresses at which most of the author's tests were made. Consequently, the results of these tests are compatible with Eq. 16 if they fall within the narrow range between the two curves for b/h = 19 and b/h = 125—which is indeed the case for high and medium stresses, indicating very good agreement between the two investigations. In the range of the very lowest stresses investigated by the author, his B_e-values seem to be somewhat larger

[17] "Tensile and Compressive Properties of Some Stainless-Steel Sheets," by C. S. Aitchison, Walter Ramberg, L. B. Tuckerman, and Herbert L. Whittemore, *Journal of Research*, National Bureau of Standards, Vol. 28, 1942, p. 499.

than the writer's. It should be noted, though, that very few of Mr. Wimer's great number of tests refer to this low range, whereas all the writer's results were obtained at low and medium stresses. The writer is inclined, therefore, to regard Eq. 16 as more generally valid than Eq. 7, because: (*a*) Both equations give very similar results in the range of most of Mr. Wimer's tests; (*b*) Eq. 16 is extensively supported by tests[14,15,16] in that range of low stresses which is scarcely covered by the author's experimental data; and (*c*) certain theoretical considerations, also discussed previously,[14,15,16] seem to favor an expression of the type of Eq. 16.

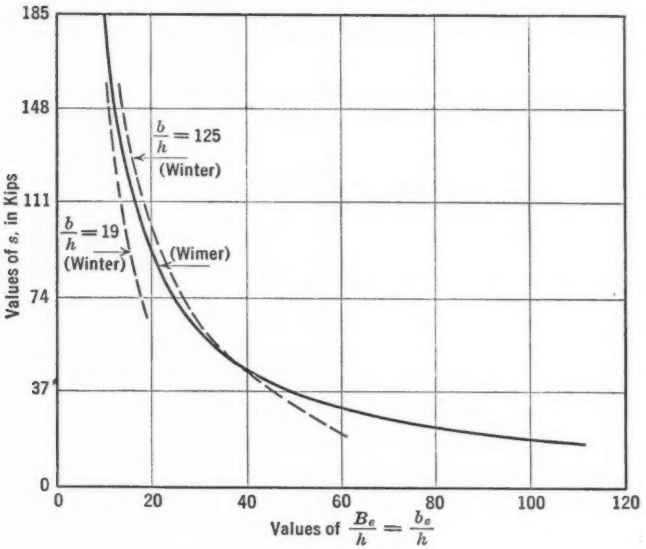

FIG. 12

It is gratifying indeed to find such excellent agreement between the two investigations over most of the range despite the fact that Mr. Wimer's tests and the writer's tests were carried out by entirely different methods and on fundamentally different materials.

The author's manner of plotting Figs. 5 and 6 could easily be understood to imply that the magnitude of the effective width depends on the ultimate tensile strength (UTS), since these graphs happen to be plotted in terms of $\sigma = s/\text{UTS}$. Actually, the effective width is a function of s/E, rather than of s/UTS, as is evident from Eq. 16 and from similar expressions available in the literature on this subject. The edge stress, to be sure, has the compression strength of the material as its upper limit, but at any stress below that limit the effective width is independent of the strength of the metal. (Unfortunately, no definition of compression strength for materials of the type of hard steels like 18–8 has yet been developed. The properties of these steels in compression and in tension are sufficiently different[17] to make the use of a tensile property in the analysis of compression tests somewhat meaningless.)

Fig. 6 could easily be interpreted as being applicable to steels other than 18–8, simply by substituting the magnitudes of their respective tensile strengths in the determination of the value of σ with which to enter the graph. To be sure, the author has not suggested that such use be made of Fig. 6. To guard against such inadvertent misunderstanding, Fig. 12 is plotted in terms of s rather than of σ. These graphs, therefore, apply strictly to 18–8 steel only; Eqs. 15 and 16, however, are quite general if proper values of E and s are sub-

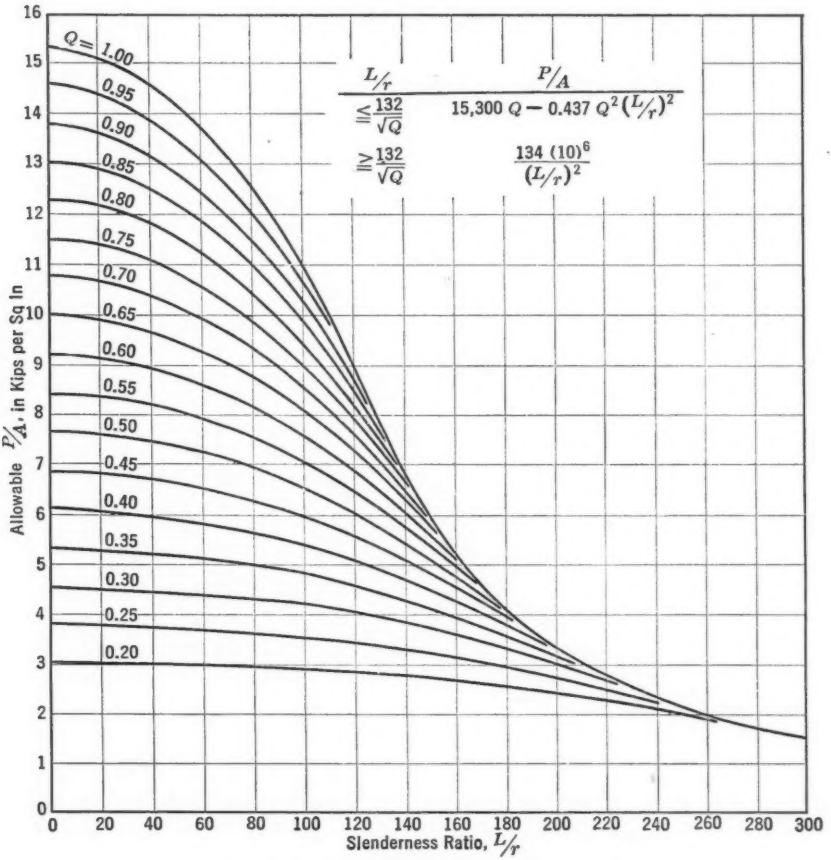

Fig. 13.—Column Design Curves for Grade C Steel

stituted. A convenient design graph derived from these equations has been published elsewhere.[15,16]

The method of modifying column formulas to account for local buckling, as described in the paper, by this time has also found its application in standard structural design specifications. In fact, the A.I.S.I. 1946 specification[11] stipulates allowable column stresses exactly in the manner indicated by the author. The column curves from this specification (Fig. 13) are precisely the

same kind as those of Fig. 8. The "form factor" Q for this chart is defined in the A.I.S.I. specification as the ratio of the effective area to the gross area, the former to be computed either for the yield stress, or for the buckling stress of the most unstable unstiffened element of the section, whichever is smaller. Thus, the physical meaning and the use of this factor are wholly identical with the author's method of substituting in a Johnson column formula the average crippling stress for the yield stress. This method, in its application to the specification, was developed jointly by J. H. Cissel, M. ASCE, F. E. Fahy, Assoc. M. ASCE, and the writer. It should be noted that Fig. 13 gives allowable stresses for mild structural steel, whereas Fig. 8 gives failure stresses for 18–8 hard steel.

Summarizing, the writer believes that:

(1) The methods discussed in this paper are not new to structural designers in the field of civil engineering. In fact, they are incorporated to various degrees in several current standard design specifications; and

(2) The paper provides valuable empirical data in the field of high-strength alloy steels. These data, obtained on 18–8 stainless steel and formulated in equations valid for such steel only, are in substantial agreement with the writer's earlier tests on mild steel members, published some years ago in a form which is applicable to almost any metallic material.

Correction for *Transactions:* In May, 1948, *Proceedings*, page 665, change the second term, s_y, of Eq. 10*b* to s^2_y.

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DISCUSSIONS

THE SIGNIFICANCE OF PORE PRESSURE IN HYDRAULIC STRUCTURES

Discussion

BY FREDERICK L. HOTES, R. W. CARLSON AND RAYMOND E. DAVIS, AND W. G. MORRISON

FREDERICK L. HOTES,[46] JUN. ASCE.—Mr. Harza has attempted to establish a revised concept of pore pressures in hydraulic structures, particularly in concrete dams. Many of his comments are quite timely, even though perhaps somewhat controversial, but the writer wishes to discuss only that part of the paper dealing with the area of application of uplift forces to a horizontal section of a dam.

The author maintains (under the heading, "Experimental Evidence") that the area subject to uplift pressure in a submerged dam is "* * * exactly 100% of the area, and no less * * *." If only the author had used the words "substantially 100%" and omitted the "and no less" the writer would heartily agree for all practical purposes with his conclusion regarding this point. However, as the statement stands it can and should be challenged, and the writer submits the following as proof that the area subjected to uplift must be somewhat less than 100%.

Without debating the question as to whether or not all the pores in a concrete dam are connected, it will be assumed that they are. If so, a concrete mass submerged in water is analogous to a mass of sand or gravel, in that all pores are interconnected and filled with water and therefore each individual particle is subjected to buoyant forces. The writer must agree with the author at this point in his statement that the popular acceptance of the effect of buoyancy on such a submerged mass (Fig. 28) is that each particle is reduced in weight by the full extent of its displacement in water. However, popular acceptance is by no means proof.

If it be conceded that this is a very close approximation to the truth the resultant force acting on a submerged particle is a downward force equal to the

NOTE.—This paper by L. F. Harza was published in the December, 1947, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: June, 1948, by William P. Creager, J. S. Kendrick, Adolf A. Meyer, E. Montford Fucik, A. C. R. Albery, and W. H. R. Nimmo; September, 1948, by John S. Cotton, Serge Leliavsky Bey, A. H. Davison, R. E. Ballester, John S. McNown, and V. T. Boughton; and October, 1948, by Ross M. Riegel, and Douglas McHenry.

[46] Lecturer in Civ. Eng., Univ. of California, Berkeley, Calif.

weight of the particle less the weight of the volume of displaced water. Fig. 29 is a free-body sketch of an individual unloaded particle and shows that an upward reaction is needed for equilibrium to be maintained. Any reaction is merely the product of a stress times an area (R being the stress times the area on which it is applied). Consequently, there must be a state of stress

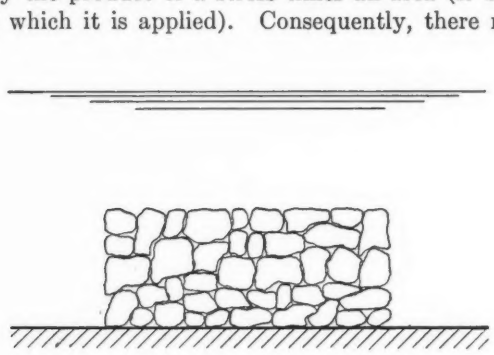

FIG. 28

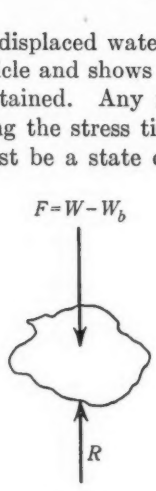

FIG. 29

and a finite area of the particle that is not exposed to water. That this stress intensity is relatively high and that the area is quite small should be realized, although the resulting extreme intensity of stress is of no particular significance in this problem. Inasmuch as a finite area does exist, the area of the particle subjected to uplift cannot be 100%, but must be something less, even though closely approaching 100%.

Within the mass of a concrete dam additional compressive or tensile forces may act on a particle, and these forces themselves must be applied by means of a stress and an area. Certain parts of a horizontal cross section of a dam may have uplift pressures acting over the entire area—for example, where the forces act upward and the base is cracked—but, as the resultant of all forces acting on a well-designed dam is downward at any such section, the conditions of equilibrium can be satisfied only by many small areas of contact whose sum in itself will be quite small. In an earth dam this contact area is close to 0%, and in a concrete dam about 1%, of the entire base area. This latter value is based on the experiments of K. Terzaghi, Hon. M. ASCE, referred to by the author.[7]

Although exception has been taken to the "exactly 100%" theory from a technical standpoint, the writer believes that for practical purposes it is applicable to all cases where full buoyancy, or nearly full buoyancy, exists.

R. W. CARLSON,[47] ASSOC. M. ASCE, AND RAYMOND E. DAVIS,[48] M. ASCE.—There appears to be sufficient proof for the author's principal contention that the uplift pressure acts over substantially 100% of the area. Not only do laboratory tests support this contention, but simple reasoning supports it also when the make-up of concrete is considered. Although a plane section cut through concrete reveals that the majority of the area is solid, a "billowy" sec-

[7] "Simple Tests Determine Hydrostatic Uplift," by Karl Terzaghi, *Engineering News-Record*, June 18, 1936, p. 872.

[47] Cons. Engr., Berkeley, Calif.

[48] Prof., Civ. Eng., and Director, Eng. Materials Laboratory, Univ. of California, Berkeley, Calif.

tion can be found near the plane section and around all the aggregate particles so that nearly all the section consists of voids. This discussion presents evidence that: (1) Pore pressure is likely never to develop to a serious intensity in continuous concrete, and (2) if pore pressure does develop in continuous concrete it is not a source of danger.

Mr. Harza's unintentional implications as to the nature of concrete tend to underrate its high degree of impermeability. His statement that "most of the pores in the cement paste will obviously be comparable in size with the cement grains" may be misleading, yet it certainly is true before the cement has hydrated. After hydration, however, part of each cement grain has been dissolved and reformed in all the water-filled interstices into a strong network of rigid "gel," with smaller amounts of solid crystalline by-products. In a cross section of hardened cement paste the gel appears solid, even under the best microscope. By indirect methods, it can be shown that the apparently solid gel is indeed porous but that the pores are from about 0.1 microinch to several microinches in size. These pores are very small as compared with cement grains, which average nearly 1,000 microinches in diameter. The effect of hydration in reducing the size of pores is so great that the permeability of concrete before hydration is about 100,000 times as great as after hydration. These relative permeabilities were measured by the U. S. Bureau of Reclamation.[3]

There is some evidence that, in a homogeneous concrete free from cracks, the pore pressure at any considerable distance from the upstream face of a dam may be negligible, at least under the conditions where the downstream face is subject to more or less continuous drying. This is indicated by the results of tests in progress in the Engineering Materials Laboratory of the University of California at Berkeley during the period from 1941 to 1948. The specimens were concrete cylinders, 8 ft long, cast in thin copper molds. After hardening, the cylinders were wound with steel wire under tension to produce a radial pressure of about 800 lb per sq in. between the copper mold and the concrete cylinder. Before winding, small capillary tubes were fixed at intervals along each cylinder in $\frac{1}{4}$-in. radial core holes of a depth equal to the radius of the specimen. Each of these capillary tubes was attached to a Bourdon tube, by which pressures were determined through observations with a micrometer microscope. Before the beginning of the test each unit, composed of a capillary tube, core hole, and Bourdon tube, was filled with water so that the Bourdon tube would immediately respond to changes in pore pressure within the concrete. The specimens were stored in a room of constant temperature (70° F) and constant relative humidity (50%), and a water pressure of 100 lb per sq in. was continuously applied to one end of each specimen. The radial pressure between mold and cylinder prevented water from percolating along this surface. For specimen 1, the opposite end was enclosed in a tightly fitting, although not airtight, metal box; for specimens 2 and 3, opposite ends were exposed to the air of the room.

[3] "An Investigation of the Permeability of Mass Concrete with Particular Reference to Boulder Dam," by Arthur Ruettgers, E. N. Vidal, and S. P. Wing, *Proceedings*, Am. Concrete Inst., March–April, 1935, p. 382.

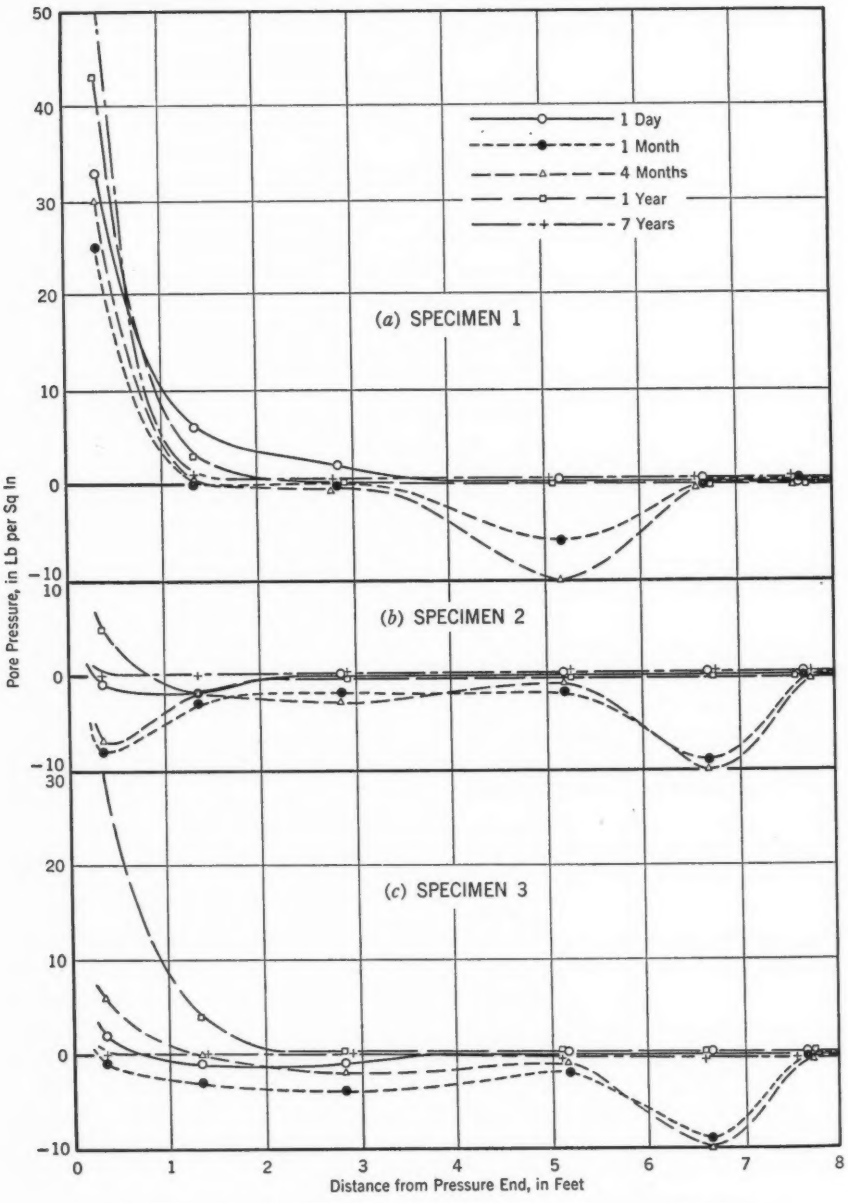

FIG. 30.—PORE PRESSURE IN 6-IN.-DIAMETER BY 8-FT-LONG CONCRETE CYLINDERS

The pore pressures observed at various ages after the beginning of the test are shown in Fig. 30. It will be observed that at one month and four months, for all three specimens, negative pore pressures (produced by continued hydration of the cement) were indicated by most of the gages. For specimens 2 and 3, the maximum negative pressure was in the order of 9 lb per sq in.

At later ages these negative pressures were relieved; but, at distances in excess of 2 ft from the end to which the water pressure was applied, in no case was the positive pressure of appreciable magnitude, and at a distance of only 4 in. from the end to which the pressure was applied, the magnitude of the pore pressure did not in any way approach that of the external water pressure.

It is interesting to note that for specimen 1, even though the pore pressures at a point 4 in. from the pressure end increased with time, at the age of seven years the positive pore pressure at all other points along the specimen had decreased to zero. Evaporation from the downstream end of this specimen could not take place as freely as for the other two specimens because of its "metal box" enclosure.

For specimens 2 and 3, for which the downstream ends were exposed to air of 50% relative humidity, the maximum pore pressure 4 in. from the upstream end developed after about one year, and at this age the pore pressures as indicated by all other gages were substantially zero. For specimen 2, after the age of two years and four months, the observed pore pressures were zero for all gages, including the one 4 in. from the upstream face. After the age of four years, for specimen 3, the pore pressure was zero at all gages.

Although the scope of the tests was too meager to justify any sweeping conclusions as to what might take place within the uncracked mass of a concrete dam, the results seem to indicate that the pore pressure at an appreciable distance from the upstream face may be quite negligible in amount and that it may not increase with time, as has generally been supposed, but under certain conditions at the later ages it may actually decrease with time. An important detail of these tests was the rather high water-cement ratio, 0.80 by weight, which, in mass concrete where large aggregate is employed, roughly corresponds to a cement content of only about 0.7 bbl per cu yd.

Even if pore pressure of high intensity should develop in the interior of continuous concrete, the effect on stability would not be large. The author's principle *e*, under the heading "Buoyancy," states in effect that the only difference between loose granular material and finished concrete is "* * * the reduction in the percentage and size of voids." This statement implies that the solidifying of concrete might actually conform to a common view that crystals or colloidal particles merely form in the interstices to change plastic concrete into hardened concrete. Actually, this view contradicts one of the most remarkable features of the hardening of concrete. The writers prefer to believe that the cement grains are dissolved partly and reformed into the rigid gel (instead of crystals), thus resulting in a strong framework around all the aggregate particles. In fact, the framework is so strong that the average tensile strength of a specimen is several hundred pounds per square inch, even though the framework makes up only a small part of the area. In general, the tensile

strength of the skeleton within which the pore pressure acts is amply great as compared with the pore pressure.

Where the concrete is not continuous due to the occurrence of open joints or cracks subject to water pressure, it is obvious that 100% uplift will exist and that no reliance can be placed on tensile strength. This is also likely to be true of the surface of contact between dam and foundation rock. However, the treatment should be to drain such regions, including the foundation, rather than to design for 100% uplift.

The subject of drainage is interesting but is not under discussion. The principal contention of the writers is that the continuous sections of a concrete dam cannot be endangered by uplift.

W. G. MORRISON,[49] M. ASCE.—Apparently, the author of this important contribution to the subject of uplift under dams expects to meet acute disagreement. It is the writer's opinion, however, that most engineers will agree cordially with practically all his arguments.

First, it would appear to be axiomatic that water pressure under a dam does apply to 100% of the area. The point that has been missed, or that certainly has not been stressed, is that when the water pressure is less than the total pressure over an area, it exists only as a contribution to the total pressure and its magnitude is of no consequence.

For example, consider a basement sunk 20 ft below ground-water level. The uplift on the base due to water pressure will be approximately 1,250 lb per sq ft. If the structure is light, or if it is assumed that the walls or footings transmit the load to a solid stratum, it may well be that the basement floor will have to be designed for 1,250 lb per sq ft less its own local weight. If, however, the basement floor is made to act as a raft for which the average load is greater than 1,250 lb per sq ft, it does not matter in the slightest whether or not full water pressure is assumed. Thus, if the average downward load is 3,750 lb per sq ft, the floor would reasonably be designed for a uniform reaction of 3,750 lb per sq ft. If the weight of the slab is 250 lb per sq ft, the uplift causing bending stresses in the slab will be 3,500 lb per sq ft, which is made up of soil reaction plus water pressure, and it does not concern the designer whether 1,250 lb or any other proportion of the reaction is due to water pressure.

Full hydrostatic head may be regarded as an inevitable component of downward pressure, and equally so of upward reaction. Provided it is less than total pressure it is not a factor causing instability, and it seems axiomatic that it should be applied to the whole surface whether caused by the water within the concrete or by the water surrounding the concrete. As in the basement illustration or in the case of a ship grounded on a mudbank on a falling tide, it is simply one component of the upward reaction.

As applied to dams, Fig. 31(*a*) shows a dam with a base width of 0.846 *H*. For a 1-ft section of the dam, the total horizontal water pressure *P* compounded with the dam weight gives *R*, which intersects the base at point x. The horizontal component must be resisted by the shear *S*, whereas the vertical component *W′* must be resisted by upward reactions. Assuming straight line

[49] Civ. and Structural Engr., Mitchell and Mitchell, Wellington, C. I., New Zealand.

variation of base pressures, the intensity of upward reactions to maintain equilibrium must vary from 6.35 kips per sq ft at the heel to 8.69 kips per sq ft at the toe. Full hydrostatic uplift amounts to 6.25 kips per sq ft, so that if and where present it is only a component of the upward reaction. Although it may exist over 100% of the base, it is of no consequence statically.

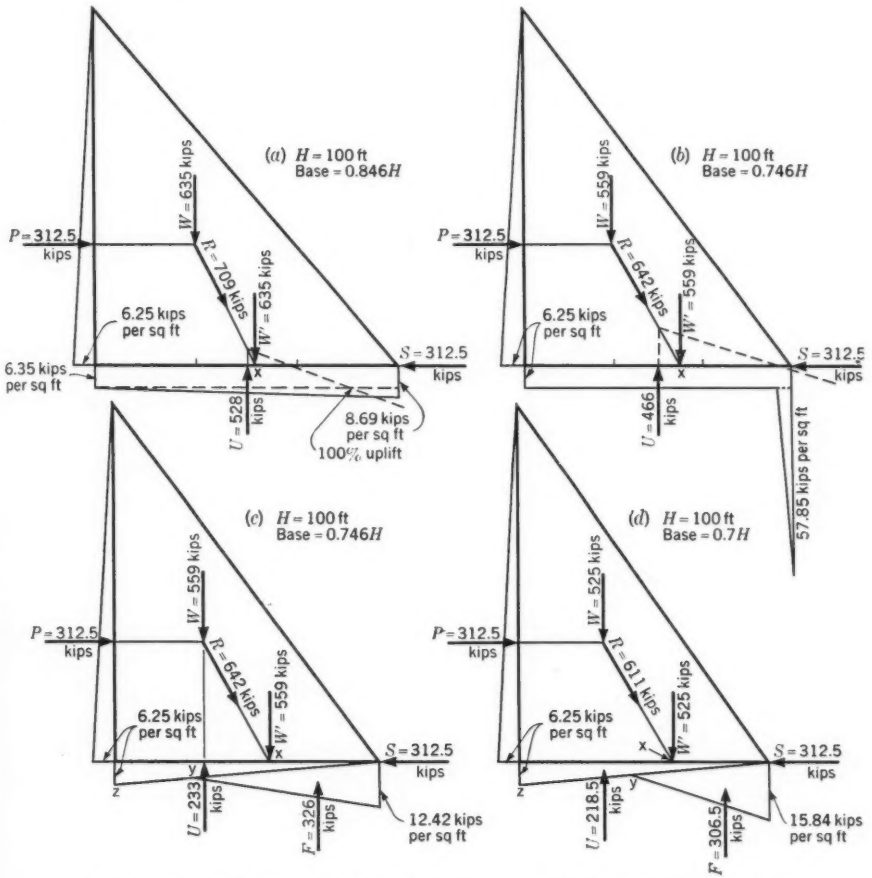
(a) H = 100 ft
Base = 0.846H
W = 635 kips
P = 312.5 kips
R = 709 kips
W' = 635 kips
6.25 kips per sq ft
6.35 kips per sq ft
S = 312.5 kips
U = 528 kips
8.69 kips per sq ft
100% uplift
(b) H = 100 ft
Base = 0.746H
W = 559 kips
P = 312.5 kips
R = 642 kips
W' = 559 kips
6.25 kips per sq ft
S = 312.5 kips
U = 466 kips
57.85 kips per sq ft
(c) H = 100 ft
Base = 0.746H
W = 559 kips
P = 312.5 kips
R = 642 kips
W' = 559 kips
6.25 kips per sq ft
S = 312.5 kips
U = 233 kips
F = 326 kips
12.42 kips per sq ft
(d) H = 100 ft
Base = 0.7H
W = 525 kips
P = 312.5 kips
R = 611 kips
W' = 525 kips
6.25 kips per sq ft
S = 312.5 kips
U = 218.5 kips
F = 306.5 kips
15.84 kips per sq ft

FIG. 31.—FORCES ACTING ON A 1-FT SECTION OF A 100-FT-HIGH TRIANGULAR DAM UNDER VARIOUS CONDITIONS

Fig. 31(*b*) shows a dam with a base width of 0.746 *H*. Again, *P*, is compounded with *W* to meet the base at point x. Now, if the uplift *U* is assumed to act with full intensity over the entire base, equilibrium can only be maintained by an upward force as shown at the toe, having a maximum intensity of 51.6 kips per sq ft over and above the water pressure, or a total of 57.85 kips per sq ft. This pressure is 25.8 long tons per sq ft on the rock, or 402 lb per sq in. on the concrete. In neither case are the pressures likely to cause failure. The dam is not unstable. Shearing stresses on the base would be too high, but an embedment of a few feet in rock at the toe would make a great difference to safety.

Fig. 31(*c*) shows the same dam as in Fig. 31(*b*) but one in which hydrostatic uplift, U, is assumed to vary from 100% at the heel to 0 at the toe. Again, the downward force at base level is W', resisted by a known uplift U and a force F, which must be calculated. Assuming triangular distribution, F gives a pressure of 12.42 kips per sq ft at the toe, and is throughout additive to the water uplift pressure. Beyond point y it does not matter what contribution to the total base reaction is due to water pressure. The important point is that water pressure from point z to point y must be reduced to the value shown by grouting, drainage, or any other expedient. If water pressure in this region builds up to 100%, the dam need not fail, but it will progressively work back to the condition shown in Fig. 31(*b*).

Fig. 31(*d*) shows a dam with a base width of 0.7 H. The condition is as shown for uplift pressure reducing on a triangular basis from point z to point y. Again it does not matter what the water pressure beyond point y may be, as long as it does not exceed the total reaction. In this case, 100% water pressure on the whole base would create a condition of instability.

These examples are intended to show that:

1. The classical "middle third" theory cannot be applied as a necessary condition of stability;
2. Stability is possible when the reaction falls outside the middle third;
3. Water uplift is only of importance as part of a total reaction; and
4. If uplift can be reduced on a triangular basis for the upstream half of the dam, it does not matter what magnitude it is for the remainder of the base, as it is only part of the total reaction.

It is realized that the foregoing examples represent an oversimplification, and that the magnitudes of reactions under dams do not follow a straight line. However, it is certainly true that the elementary principles of static equilibrium must be observed in all cases.

In explanation of why some dams having bases narrower than 0.7 H have not failed, the writer would suggest these possibilities:

a. The reduction of the magnitude of the uplift pressure under the heel, as in Fig. 31(*d*), from point z to point y (this in accordance with the design, and the measures taken to achieve it);

b. The greater weight of the concrete near the water face because of the greater degree of saturation;

c. Friction between the blocks of the dam, with the more stable segments near the ends of the dam;

d. The fact that toe pressures much greater than those normally assumed may exist (Fig. 31(*b*)); and

e. For dams only occasionally filled there may even be "limpet" action due to air pressure on the back, and, through the medium of the water, on the front. The resultant will be a contribution to stability unless and until the effect is communicated under the base by seepage, which will take time. In any case, the writer would discount tension between the dam and the base as most unlikely and very unreliable.

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Founded November 5, 1852

DISCUSSIONS

THE RELATIONSHIP BETWEEN PILE FORMULAS AND LOAD TESTS

Discussion

BY JACOB FELD, RICHARD F. KAFKA, G. B. R. PIMM, AND GEORGE D. HARTLEY

JACOB FELD,[23] M. ASCE.—This excellent critical analysis of the correlation between the ultimate load value of piles as given by load tests and the theoretical ultimate value as given by the several pile-driving formulas is based on the assumption that the load tests give true results. Failure of a pile in cohesionless soil under the slowly increasing load normally applied in a structure, if it is carried to the ultimate value, probably occurs under a greater load than is determined from a pile load test extending over one day, or at the most, two days. Load tests on piles in cohesive soils, however, often indicate safe values which prove excessive in the actual structures. Even in saturated silty soils, 18-in. pipe piles which passed a load test of 200 tons, after being driven by a No. 1 Vulcan hammer to a "safe" value of more than 100 tons as determined by the Engineering-News formula, were found to have a driving resistance of much smaller values one month later.

The author convincingly proves the superiority of the Hiley type of formula over the older types, and very properly explains that Arthur M. Wellington, M. ASCE, never intended that his formula be employed for the type and length of piles now being driven, or for the large impact energies of modern hammers. It is to be noted that the maximum safe carrying capacity of piles in tons which can be determined by the application of the Engineering-News formulas is 0.001 times the energy of the blow in foot-pounds for drop and single-acting hammers, and 0.01 times the rated energy in foot-pounds for double-acting hammers. In these cases, the set per blow is zero. This means that a double-acting hammer of 10,000-ft-lb rated energy cannot produce a pile value of more than 100 tons, and a drop hammer weighing 3,000 lb and dropping 5 ft cannot produce a pile value of more than 15 tons. Many more inconsistencies can be shown in these formulas, yet they served well and for many years. Certainly, their application should be limited, as is customary among the engineers practicing in the New

NOTE.—This paper by Robert D. Chellis was published in May, 1948, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1948, by Donovan H. Lee.

[23] Cons. Engr., New York, N. Y.

York (N. Y.) area, to pile loads not more than 40 tons (some limit further to 30 tons) and in soils which are predominantly granular.

The more serious problem is to find a true guide for values of piles driven in plastic or viscous soils. No energy resistance formula gives reliable values; load tests also disregard the important factors of time and loading rate. The author's suggestion that surface shear resistance be computed as a guide and check on load tests is probably as good a method as the present state of knowledge permits. The surface shear resistance determined at failure is smaller than the latent shear resistance when no motion occurs. However, a long time load test will show a creep settlement and there are no available data on the relationship between the amount of creep (settlement), the percentage of maximum shear resistance being balanced by the load, and the time. Another serious gap in necessary values is the reliability of simultaneous action of end bearing and surface friction resistance. Many of these missing data must be determined before a complete solution of the pile-driving formula becomes possible.

RICHARD F. KAFKA.[24]—It gave the writer great satisfaction to read this paper, as it reminded him in some respects of his own studies on this subject, around 1908. The determination of the loading capacity of driven piles was rather a dark field then and it is still not entirely clarified as is apparent from Mr. Chellis' paper.

First, it should be stressed that a structural member which carries the loads to rock or to a stratum that acts like rock is not a pile from the point of view of statics, even if it has been driven like a pile. It is, rather, a column. The only difference between a genuine column and a pile-like column or end-bearing pile is that the latter is surrounded by earth, which prevents it from buckling and thus increases the load capacity. If the rock at and below the tip of the pile actually exists and the pile is capable of transmitting the load to this stratum, all conditions required for an effective foundation are fulfilled as far as statics is concerned. In such cases, no driving formula need be applied to determine the loading capacity.

Thus, excluding column-like piles from the discussion and considering only genuine piles, the writer agrees with Mr. Chellis that loading tests are the most reliable method of determining the ultimate and the permissible load of a pile. However, engineers have need of a quick and simple method of determining the loading capacity and thus have been led automatically, so to speak, to the application of driving formulas or dynamic formulas.

Among the numerous driving formulas commonly used, those must be disregarded which are based on impossible assumptions (for example, on perfectly elastic or perfectly inelastic pile and hammer) or which neglect the elasticity (if any) of the soil. Driving formulas can be expected to give true load-bearing values only if applied to piles driven into cohesionless soil, as mentioned by the author, under the heading "Dynamic Formulas." In developing a driving formula published elsewhere,[25] the writer was well aware of

[24] Editor, *Engineering Index, Inc.*, New York, N. Y.

[25] "Die Theorie der Pfahlgruendungen," by Richard F. Kafka, J. Springer, Berlin, 1912.

this fact, as well as that approximate assumptions of the elastic properties, both of the pile and the soil, constitute the weak point of all driving formulas. The only possibility of overcoming conditions which can make even the most scientific driving formulas valueless is the measurement of the elastic deformation caused by the hammer blow. This method, which was first suggested by M. Genel and M. Konrad, has been treated extensively, and an apparatus has been described[25] by which the measurement can be accomplished. The measurement has the advantage that all variables of pile, hammer, and soil, including friction, air resistance, and other factors, which affect the driving results are taken into consideration automatically. Mr. Chellis has stated (under the heading, "Comparison of Driving and Test Load Values") that the relationship between set and elastic compression changes not only with each pile, but also with each hammer blow; hence, no formula can be expected to apply to all conditions. The measurement of the sets and the elastic compression also solves this problem and there is no reason to exclude the application of driving formulas to cohesive soils.

G. B. R. Pimm.[26]—In discussing dynamic formulas the author states, under the heading, "Dynamic Formulas," that it is a basic error to consider the value of l as the full length of the pile regardless of the location of the center of driving resistance. In using the Hiley formula the value of C, unless actually measured, should therefore be calculated from l as defined by the author. The same point is made, under the heading, "Failure Point in Load Tests," in considering the elastic deformation under static loading. However, it is difficult to ascertain the position of the center of driving resistance in the absence of full information as to the proportions of the total resistance caused by frictional resistance and toe resistance, respectively.

There is another aspect of the question of frictional support that is of even greater importance than its effect on the elastic strain. When a pile is partly supported by friction, there may be an elastic movement of the ground material throughout the frictionally supported length. The weight set in motion by each hammer blow is then not merely W_p but $W_p + W_g$, in which W_g is the weight of an unknown quantity of ground material which moves in sympathy with the pile and absorbs an unknown proportion of the energy of the blow. The omission of this factor is a basic defect in all formulas containing the ratio $\frac{W_p}{W_r}$ or any modification thereof, such as the Hiley, Eytelwein, Pacific Coast Uniform Building Code, Canadian National Building Code, and many other formulas. In the Hiley formula (Eqs. 6 and 7), for example, the efficiency is not $\frac{W_r + e^2 W_p}{W_r + W_p}$, but $\frac{W_r + e^2 (W_p + W_g)}{W_r + (W_p + W_g)}$.

The errors due to (1) taking the full length instead of the effective length, and (2) neglecting the energy expended in moving the weight W_g are of opposite sign, and therefore tend to cancel out, but in any given case in which one or the other predominates, the result may be seriously affected. In both cases the error has its origin in the frictional support, which must be eliminated before

[26] Central Chambers, Ealing, London, England.

the toe resistance can be arrived at by an impact formula. To achieve this, recent British practice has been to carry out preliminary tests by the prepiling method, in which the toe resistance is recorded independently of the frictional resistance at all material depths, and is interpreted by the Hiley formula. The apparatus consists of an outer tube and an inner mandrel, the latter being driven separately (thus eliminating skin resistance) at those points in depth at which it is desired to ascertain the true toe resistance. Between these points the assemblage is driven as a whole, like an ordinary pile, against the total resistance; but, since the calculation of the latter, from the driving record, would entail the reintroduction of the error referred to previously, the frictional resistance is found separately, by dead loading or by jacking.

In this way, the two elements of the resistance are accurately ascertained, each by an appropriate method. Experience has shown, however, that the frictional resistance is a very capricious factor—an additional reason for eliminating it from the calculation of the more dependable toe resistance.

GEORGE D. HARTLEY,[27] ASSOC. M. ASCE.—By drawing on a wealth of experience obtained from a wide variety of pile-driving records, Mr. Chellis has

TABLE 5.—NUMBER OF BLOWS REQUIRED FOR A PENETRATION OF 1 FT

Depth of pile below ground (ft)	PILE NO.								
	219	220	221	222	223	224	225	239[a]	240
5	15	15	10	19	....	19	20	33	15
6	....	....	15	15	16	....	20	40	16
7	....	....	19	15	....	....	21	40	16
8	....	....	15	19	....	....	20	44	21
9	....	....	16	20	....	....	20	60	25
10	33	21	17	23	20	25	29	58	30
11	....	....	16	19	....	....	37	59	30
12	....	....	14	21	....	....	40	60	30
13	....	....	12	18	....	....	36	60	29
14	....	....	13	14	....	....	23	56	22
15	41	25	11	13	21	25	17	56	19
16	....	....	10	10	....	....	15	33	12
17	....	....	9	10	....	....	14	34	12
18	....	....	7	10	....	....	13	34	12
19	....	....	7	7	....	....	13	33	11
20	18	17	8	8	13	16	13	35	11
21	....	....	7	8	....	....	12	....	10
22	....	....	8	9	....	....	13	....	10
23	....	....	7	9	....	....	14	....	11
24	....	....	7	10	....	....	15	....	12
25	15	16	7	11	16	19	18	30	14
26	....	....	8	11	....	...	17	25	15
27	....	....	9	13	....	....	18	33	17
28	18	18	13	14	14	20	17	32	20
29	18	17	15	16	15	18	21	30	....
30	23	20	15	....	17	20	25	29	....

[a] Pile No. 239 was 50 ft long, but only values to 30 ft have been shown for comparative purposes.

shown the lack of positive correlation between pile capacities derived by the use of a standard formula and pile capacities as obtained by test loadings. On a recent project capacities of piles driven in Norfolk, Va., were based on the results of test loadings applied to two piles. Although the soil was not composed

[27] Supt. of Constr., Doyle and Russell, Gen. Contractors, Richmond, Va.

of truly cohesionless materials, but showed intense stratification, it is thought that the results obtained may be of interest.

The building code (1948) of the City of Norfolk provides that piles be driven in accordance with the Engineering-News formula. As shown in Table 5, which is the driving log for a number of piles, erratic driving conditions were encountered, as illustrated by the fact that pile No. 240 required approximately only one half of the number of blows required by pile No. 239 to obtain the same penetration although the two piles were driven only 2 ft 6 in. apart. This condition could not be traced to any unusual conclusions based on pile-driving operations, as there was a time lapse of one month between the driving of these piles, pile No. 239 being driven first. As a further indication of comparative driving resistance it may be interesting to note that piles Nos. 219 to 225, inclusive, were driven within a 6-ft radius from load test pile No. 222.

In view of this experience with erratic driving conditions, it was decided to make loading tests on two of the piles driven—Nos. 222 and 240. These piles were selected because they showed the most variance on the basis of allowable load from the design load of 15 tons based on the Engineering-News formula. The load tests were conducted in strict accordance with the City of Norfolk Building Code (1948), which required that the following conditions be satisfied:

> "* * * Safe load carrying capacities may be determined by means of loading tests performed on not less than two typical piles of an installation. The allowable load shall in no case exceed one-half of that load which causes a net settlement (total settlement less rebound) of 1/200 of an inch per ton of test load or more than one-half inch, nor shall it exceed one-half of that load under which, during a 48 hour period of continuous constant load application, no appreciable additional settlement takes place."

TABLE 6.—LOAD TEST DATA FOR PILES NO. 222 AND NO. 240

Time (min) (1)	Load (tons) (2)	Settlement (in.) (3)	Time (min) (1)	Load (tons) (2)	Settlement (in.) (3)
(a) PILE NO. 222			(b) PILE NO. 240		
0	0	0	0	0	0
50	6	0	23	6	0
73	12	0	52	12	1/32
86	15	0	73	15	1/32
96	18	0	81	18	1/32
99	20	0	89	20	1/32
117	24	1/32	103	24	1/16
124	25.5	1/16	118	27	1/32
142	30	1/4	133	30	1/4
48[a]	30	1/4	48[a]	30	1/32
90[a]	0	1/16[b]	90[a]	0	1/16[c]

[a] Hours. [b] Rebound 1/16 in. [c] Rebound 1/32 in.

As shown in Table 6 the load test on these piles easily satisfied the test load requirements of the building code. The allowable load capacity as derived from the Engineering-News formula is 8.54 tons for pile No. 222, and 12.1 tons for pile No. 240. The ultimate capacities based on a safety factor of 6 would

indicate values of 51.24 tons and 72.6 tons for piles Nos. 222 and 240, respectively. Design loads for the piles under consideration were based on an allowable load of 15 tons, which was indicated as permissible as a result of the load tests. The test piles were not loaded to failure and the actual safety factor applicable to the results obtained by the load test is not known. It may possibly be that, in dealing with two separate provisions of a building code (pile capacity based on formula and pile capacity based on the results of load tests), there are involved two variable safety factors each separate and distinct from the other.

As piles were driven and it was noticed that the erratic driving conditions continued, a check was made on all variables which might account for the discrepancies. However, no positive correlation between these factors and the penetration per blow could be determined. The factors observed were: Size of tip and butt, steam pressure, rate of blows, and order of driving. Piles were driven with a single-acting steam hammer.

Relatively speaking this was a small job, but the cost of the load tests was a justified expense that was more than offset by the saving in pile costs.

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DISCUSSIONS

VALUATION AND DEPRECIATION OF PUBLIC UTILITY PROPERTY

Discussion

BY W. V. BURNELL

W. V. BURNELL.[27]—In the view of the writer, the author has made a significant contribution toward the clarification of a subject which has long been mired down in confusion—namely, the broad subject of fair value as related to regulated public utilities.

Governmental regulation of public utilities in the final analysis is a judicial process that calls for the formation of reasonable judgments, supported by and in harmony with acceptable evidence. The final resort in such proceedings, in the United States, is to the judicially pre-eminent Supreme Court. One looks to that tribunal, beyond all others, for a clarity of reasoning of the highest order. There is cause for discouragement, however, when that court sustains an order as reasonable but fails to disclose "what makes it so or what could possibly make it otherwise," as was charged in the dissenting opinion of Mr. Justice Robert H. Jackson in the Hope Case.

Nevertheless, that discouragement may not be so completely attributable to the shortcomings of the Court as it may appear for, after all, fairness in public utility rate making and particularly the determination of property values is fraught with many technical factors of economics as well as engineering. If some of these factors are not brought clearly to the Court's attention, it is hardly to be expected that they will be considered.

There may be ground for a belief that the engineering profession is somewhat at fault for this error of omission. It is on that account especially that Mr. Scharff's clarifying contribution deserves more than a passing word of commendation. He lays no claim to the origination of the concept of "service value," but he has presented the concept with noteworthy persuasiveness. The engineering fraternity would do well to take heed thereof. The regulatory commissions, the courts, and especially the United States Supreme Court, are entitled to better "engineering-economic" advice than they have received in the past.

NOTE.—This paper by Maurice R. Scharff was published in June, 1948, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1948, by J. Kappeyne.

[27] Vice-Pres., Stone and Webster Eng. Corp., New York, N. Y.

The writer confesses to a definite share of blame for the failure of the engineering profession to "put across" the soundness of this cost-of-service concept of value. In a case before the Public Utilities Commission of the District of Columbia, he went to great pains to demonstrate the concept as applied to the property of the Potomac Electric Power Company, but his powers of persuasion proved to be quite inadequate.

By the very practical process suggested in that case, and now by Mr. Scharff, one arrives first at a fair price for the product—the price for which a possible or potential competitor could afford to sell the product after providing for the most efficient facilities available for its production. When that result is obtained, the answer to the question of value may be found by means of a simple arithmetical equation.

Only one question is raised as to Mr. Scharff's conclusions, and that concerns only a single factor of the equation—the "annual depreciation allowance." Referring to Table 1, the conclusion for the property as a whole is a summation of the results for each of the classified groups. For simplicity, Account No. 316, coke ovens, is selected for the purpose of analysis. For the most economical substitute plant the investment would be $1,800,000 and the annuity factor at 6% for a life expectancy of 35 years is 0.06897, the amount of the annuity being $124,100. An adjustment is made here to bring this annuity down to $121,200, which, in effect, reduces the $1,800,000 to $1,757,200. For the excess cost of operation and maintenance of the existing plant there is a deduction from the foregoing annuity of $30,000, leaving a balance of $91,200 available as an annuity on the existing plant for its remaining service life, which Mr. Scharff has taken to be 25 years. Since the present worth factor, for 25 years at 6%, is 12.783, the indicated present value of this annuity, and therefore of the existing plant, is $1,165,800.

The annuity factors are very important in the foregoing calculation. They must necessarily be based on a judgment as to the respective remaining life of each of the two plants under consideration. The writer does not undertake to question either of the adopted estimates of remaining life. He does raise a question, however, as to the consistency of either estimate in its relation to the other.

For this line of inquiry the only published precedent known to the writer is a paper[28] presented at a meeting of the Technical Valuation Society in January, 1947. The present situation is especially opportune to develop in more meticulous detail the essential point of that paper.

In this case of the coke ovens, the fundamental hypothesis is that the remaining life of the existing property is 25 years. Provision is made, as a part of the annuity, for the accumulation of a 100% depreciation reserve, during the 25-year period, to compensate for the full loss in value of $1,165,800. Consistency appears to require the assumption that this future loss in value will have been caused, as a general rule, not by the physical wearing out of the property, but by developments similar to those which have brought about the present loss in value of the existing plant. These developments, both external to the existing plant, are: (*a*) A smaller amount of investment required, and (*b*) a lower

[28] *Technical Valuation*, April, 1947.

cost of operation. It follows that the prospective fall in value of $1,165,800 will be the result of a combination of these same two external factors. It also follows that the development of these factors, caused by improvements in the design of a future substitute plant, would necessarity have the same future effect on the present substitute plant as found in the case of the existing plant.

The next implication of the process is that the accumulated amount of the depreciation reserve for the present substitute plant at the end of the 25-year period, if today's assumptions of remaining life are consistent, must also be $1,165,800. What is the fact about this? The depreciation annuity factor for the substitute plant is 0.00897. Against the adjusted investment in this plant of $1,757,200 the amount of the annual depreciation charge is $15,760. From the tables it will be found that the accumulated amount of these charges will be 54.86 × $15,760, or $864,600.

It would follow from the foregoing that the remaining life assumptions of 35 years and 25 years, respectively, for the substitute and existing plants, as tested by the yardstick of prospective improvements in design, are not consistent. Assuming the 35-year assumption to be correct for the substitute plant, the calculations of the writer force the conclusion that the correct assumption for the existing plant would be slightly more than 30 years. On this basis, moreover, the economic value of the existing plant comes to $1,257,200, instead of $1,165,800. The depreciation annuity factor for 30 years (plus) is 0.01254, which produces the same $15,760 required for the substitute plant with a 35-year life. At the end of the 30 years (plus) with a multiplier of 79.77 the accumulated reserve will amount to $1,257,200 for both plants.

A similar calculation accepting the 25-year life for the existing plant results in a life expectancy for the substitute plant of approximately 29.5 years, and a value for the existing plant of the same $1,257,200. In both cases it will be seen that the difference in value between the substitute plant ($1,757,200) and the existing plant ($1,257,200) is exactly $500,000. Upon further reflection, it becomes clear that this difference in value is nothing more than the capitalization at a 6% rate of the original $30,000 annual operating expense savings.

In arriving at estimates of the total cost of service it is clearly necessary to include an adequate allowance on both sides of the equation, for a depreciation annuity. From the viewpoint of comparative property values, however, the essential requirement would seem to be that the allowances be consistent with each other.

In the over-all result as reached by Table 1, the indicated total annual saving is $190,000. It will be observed, however, that this potential saving is related to less than half of the total cost of the substitute plant. For the remainder of the property, the impairment in value must be attributed, in its entirety, to the mere lack of newness in the existing property. The extent of the indicated impairment, in each of these remaining accounts, results only from the corresponding assumptions as to the length of the capitalization period—that is, the remaining service life.

The important point to consider, in this matter of relative age, is that no provision appears to have been made in Table 1 for any saving in the annual cost of maintenance that might be gained by having a "new" property in place

of the "old" one. Unless there would be such a saving, it is difficult, with only a few special exceptions, to conceive a reasonable justification for the abandonment of the existing property for the sole reason of its physical deterioration. The exceptions relate to such items as individual lengths of pipe, wooden poles, or furnace linings, which are subject to corrosion, decay, and the ravages of intense heat, respectively.

In general it is safe to state that public utility property, or any industrial property for that matter, can be kept in physical condition for service indefinitely by adequate maintenance and replacement of parts, usually charged to the maintenance account. Does it not follow from this that the differential in value between a new property of this character and one that is "old" is, in practical reality, the capitalized value of the difference between the two in the annual cost of maintenance and physical upkeep?

Referring again to Table 1, it is suggested that Col. 9 ("Annual Operating Expense Savings by Most Economical Facilities") be amplified in its scope to include any saving in the cost of maintenance that would be gained because of the newness of the substitute property. With this amplification, the limitation in the expectancy of remaining service life would be governed entirely by external developments. The required consistency in the relative annual allowance for depreciation could thus be satisfied and the "speculative" element in the calculation, in so far as relative property value is concerned, could thus be eliminated. It is quite probable that if the author adopted the modification herein suggested his final conclusion would not be greatly changed. It is believed, however, that the conclusion would be founded on a less speculative and more provable basis.

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DISCUSSIONS

DETERMINATION OF POSITION AND AZIMUTH BY SIMPLE AND ACCURATE METHODS

Discussion

BY T. F. HICKERSON

T. F. HICKERSON,[16] M. ASCE.—As explained in the "Introduction," this paper deals primarily with the precise determination of azimuth and only incidentally with the determination of latitude and longitude. It might also be stated that in trying to simplify azimuth calculations the motive is to encourage wider use of true meridian reference lines on the part of engineers and surveyors who remember little or nothing about the manipulation of trigonometric and logarithmic functions.

In calculating latitude the discussers are correct in stating that an adaptation has been made of methods used by navigators, particularly those of A. A. Ageton.

Captain Weems, sponsor of the Weems System of Navigation, was requested to present a discussion in order to represent the navigators' point of view and thus to give credit where credit is due. Captain Weems makes a welcome contribution in presenting the background of the log secant and cosecant tables. These quantities are relatively simple in being always positive. They are beginning to appear in textbooks on trigonometry, designated by their real names, of course, and not by the letters A and B.

The author endorses Captain Weems' suggestion for uniform symbols, and expresses the hope that those used by navigators will be adopted—that is, L for latitude instead of ϕ (a well-known symbol for angle of friction), and d for declination instead of δ.

Professor Penn, on noting that the latitude determination is similar to the Ageton method, concludes that nothing in the paper is new or simplified. The writer disagrees emphatically with Professor Penn's statement that "There are simpler methods of representing the zenith-pole-object triangle than those given in Figs. 1, 2, and 3."

Mr. Wylie's program of observations for poorly mapped regions is a valuable contribution to the paper, and it includes, among other suggestions, a good resume of the ideas presented. Mr. Wylie recognizes that Eqs. 7 and 9 are

NOTE.—This paper by T. F. Hickerson was published in November, 1947, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: June, 1948, by John C. Penn, Paul E. Wylie, and P. V. H. Weems.

[16] Prof. of Applied Math., Univ. of North Carolina, Chapel Hill, N. C.

new, but he raises the question as to the advantage of the exclusive use of secants. The answer is based on a desire to promote the greatest possible simplicity.

The following equation (a well-known formula) would give the same result:

$$\tan^2 \tfrac{1}{2} Z = \frac{\sin (s - \phi) \sin (s - h_o)}{\cos s \cos (s - p)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (37)$$

However, it involves three trigonometric functions with possible negative characteristics (in case logarithms are used), whereas Eq. 9 deals entirely with one function whose values are always positive numbers greater than unity. To the expert this difference is trivial but to those who are unfamiliar with trigonometry and logarithms it assumes importance. Of course, Eqs. 9 and 37 apply mainly to solar observations where the altitudes are needed.

Eqs. 13 and 14 might be added to the list of new formulas. In comparison with Eq. 35 they possess the advantage of being adapted to logarithmic computation. As pointed out, the combinations of Eqs. 4, 5, 6, and 15 are easier to solve provided the work form given in Fig. 7 is followed. The letter h is used in these formulas only as a medium. Actually the altitudes are unnecessary.

To conform to north as zero azimuth, Eq. 35 should be rewritten

$$\tan Z = \frac{\sin t_Z}{\sin \phi \cos t_Z - \cos \phi \tan \delta} \quad \ldots\ldots\ldots\ldots\ldots\ldots (38)$$

Strict attention must be paid to the algebraic signs when using Eq. 38. If a table of natural trigonometric functions and a calculating machine are available, Eq. 38 is a very acceptable formula. As an illustration, Example 4 will be solved. Given $t_Z = -\ 49°\ 59.0'$, $\delta = 38°\ 43.9'$, and $\phi = 32°\ 05.4'$. Then $\sin t_Z = -\ 0.76586$; $\cos t_Z = 0.64301$; $\sin \phi = 0.53125$; $\cos \phi = 0.84722$; and $\tan \delta = 0.80206$. Applying these data to Eq. 38,

$$\tan Z = \frac{-\ 0.76586}{0.34160 - 0.67952} = \frac{-\ 0.76586}{-\ 0.33792} = +\ 2.26639$$

Hence $Z = 66°\ 11.5'$ NE, and $Z_N = 66°\ 11.5'$. The two calculated values of Z in Fig. 7 are $66°\ 11.6'$, which is satisfactory agreement.

The writer is glad to have unanimous approval of hour angle methods over altitude methods for determining azimuths of close circumpolar stars; and the remarks by Mr. Wylie as to the precision of time from radio signals reinforce the paper to a considerable extent.

No comment was made on Table 1. This information came by scanning the pages of *Publication No. 214* (a Public Works Administration project conducted by the Hydrographic Office of the United States Navy Department), that monumental work of nine volumes comprising the complete solution in tabular form of approximately 1,500,000 astronomical triangles. These extracts in Table 1 are very enlightening. Suppose, for example, the latitude is 20°N and the star is Capella; then, as the hour angle changes from 50° to 90°, and the corresponding altitudes change from 41.7° to 14.2°, the azimuth varies from 45.5° to 45.8°. In other words, this star appears almost stationary in direction during the interval of 3 hours 40 min. Attention is called to the fact that these hour angles apply to periods both before and after transit.